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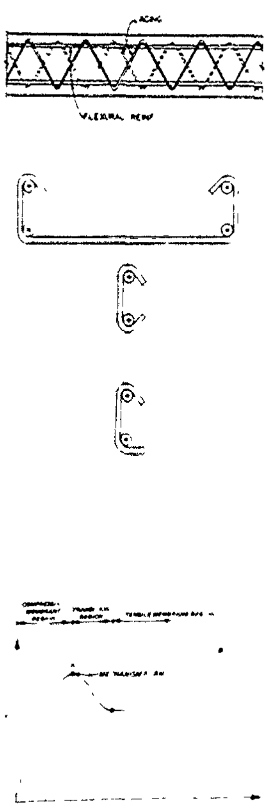
TECHNICAL REPORT SL-90-11

RESPONSE LIMITS OF BLAST-RESISTANT SLABS



by

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October 1990
Final Report

Prepared for US Army Engineer District, Omaha
Omaha, Nebraska 68102-4978

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved
OMB No 0704 0188
Exp Date Jun 30 1986

1a REPORT SECURITY CLASSIFICATION Unclassified	1b RESTRICTIVE MARKINGS
2a SECURITY CLASSIFICATION AUTHORITY	3 DISTRIBUTION/AVAILABILITY OF REPORT Approved for public release; distribution unlimited.
2b DECLASSIFICATION/DOWNGRADING SCHEDULE	
4 PERFORMING ORGANIZATION REPORT NUMBER(S) Technical Report SL-90-11	5 MONITORING ORGANIZATION REPORT NUMBER(S)
6a NAME OF PERFORMING ORGANIZATION USAEWES Structures Laboratory 6b OFFICE SYMBOL (If applicable) CEWES-SS	7a NAME OF MONITORING ORGANIZATION
6c ADDRESS (City, State, and ZIP Code) 3909 Halls Ferry Road Vicksburg, MS 39180-6199	7b ADDRESS (City, State, and ZIP Code)
8a NAME OF FUNDING/SPONSORING ORGANIZATION US Army Engineer District, Omaha 8b OFFICE SYMBOL (If applicable) CEMRO-ED-SH	9 PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
8c ADDRESS (City, State, and ZIP Code) Omaha, NE 68102-4978	10. SOURCE OF FUNDING NUMBERS PROGRAM ELEMENT NO | PROJECT NO | TASK NO | WORK UNIT ACCESSION NO

11 TITLE (Include Security Classification)
Response Limits of Blast-Resistant Slabs

12 PERSONAL AUTHOR(S)
Woodson, Stanley C

13a TYPE OF REPORT	13b TIME COVERED	14 DATE OF REPORT (Year, Month, Day)	15 PAGE COUNT
Final Report	FROM ______ TO ______	October 1990	91

16 SUPPLEMENTARY NOTATION
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161

17 COSATI CODES			18 SUBJECT TERMS (Continue on reverse if necessary and identify by block number)
FIELD	GROUP	SUB-GROUP	Ductility; Large deflections; Reinforced concrete slabs; Shear reinforcement

19 ABSTRACT (Continue on reverse if necessary and identify by block number)

The use of some type of shear reinforcement (lacing bars or stirrups) is required by current manuals for blast-resistant design. The primary purpose of this reinforcement is not to resist shear stresses, but rather to improve performance in the large-deflection region by tying the two principal reinforcement mats together. The shear reinforcement design criteria of the current design manuals, particularly the widely used Draft TM 5-1300, appear to be overly conservative. The design criteria are based on an incomplete test series. Recent tests indicate that slabs with stirrups can sustain support rotations significantly larger than allowed by the manual.

If overly conservative design criteria are eliminated, the structures can be designed with more confidence and constructed more economically than current procedures allow. Suggestions for new design guidelines are presented. Further testing and/or analyses are needed to allow validation or modification of the suggested revisions. These guidelines are primarily based on response limits. An analytical procedure for more accurately determining shear reinforcement requirements is needed.

20 DISTRIBUTION/AVAILABILITY OF ABSTRACT ☐ UNCLASSIFIED/UNLIMITED ☒ SAME AS RPT ☐ DTIC USERS	21 ABSTRACT SECURITY CLASSIFICATION Unclassified
22a NAME OF RESPONSIBLE INDIVIDUAL	22b TELEPHONE (Include Area Code) | 22c OFFICE SYMBOL

DD FORM 1473, 84 MAR — 83 APR edition may be used until exhausted. All other editions are obsolete.

SECURITY CLASSIFICATION OF THIS PAGE
Unclassified

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SECURITY CLASSIFICATION OF THIS PAGE

SECURITY CLASSIFICATION OF THIS PAGE

PREFACE

This study was conducted by the US Army Engineer Waterways Experiment Station (WES) for the US Army Engineer District, Omaha, Engineering Division, Special Projects Branch, Hardened Structures Section (CEMRO-ED-SH). The monitor at CEMRO-ED-SH was Mr. William H. Gaube.

This work was conducted at WES under the supervision of Messrs. Bryant Mather, Chief, Structures Laboratory (SL); and James T. Ballard, Assistant Chief, SL; and Dr. Jimmy P. Balsara, Chief, Structural Mechanics Division (SMD), SL. Mr. Stanley C. Woodson, SMD, performed the study and prepared this report.

COL Larry B. Fulton, EN, is Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745	radians
feet	0.3048	metres
inches	25.4	millimetres
kips (force) per square inch	6.894757	megapascals
pounds (force)	4.448222	newtons
pounds (force) per square inch	0.006894757	megapascals

RESPONSE LIMITS OF BLAST-RESISTANT SLABS

CHAPTER 1: INTRODUCTION

Background

1. Most design guides and manuals for blast-resistant reinforced concrete structures stipulate the use of shear reinforcement irrespective of shear stress levels. The primary purpose of shear reinforcement is not to resist shear forces, but rather to improve performance in the large-deflection region by tying the two principal reinforcement mats together. Shear reinforcement used in blast-resistant design usually consists of either lacing bars or single-leg stirrups (Figure 1). Lacing bars are reinforcing bars that extend in the direction parallel to the principal reinforcement and are bent into a diagonal pattern between mats of principal reinforcement. The lacing bars enclose the transverse reinforcing bars which are placed outside the principal reinforcement. The cost of using lacing reinforcement is considerably greater than that of using single-leg stirrups due to the more complicated fabrication and installation procedures.

2. In the design of conventional structures the primary purpose of shear reinforcement is to prevent the formation and propagation of diagonal tension cracks. The shear reinforcement requirements for conventional structures are based on much research and data from static beam tests. Very little study has been devoted to examining the role of shear reinforcement in slabs under distributed dynamic loads, especially in the large-deflection region of response. In blast-resistant design, structures are typically designed to survive only one loading and relatively large deflections are acceptable as long as catastrophic failure is prevented.

3. Some type of shear reinforcement in the form of lacing or stirrups is required by applicable design manuals for almost all blast resistant structures. A considerable amount of data from various tests conducted on slabs indicate that the shear reinforcement design criteria typical of current design manuals may be excessive. This data base primarily consists of slab tests conducted to investigate parameters other than shear reinforcement details. A thorough study of the role of shear reinforcement (stirrups and lacing) in structures designed to resist blast loadings or undergo large deflections has never been conducted. A better understanding of the mechanics of the behavior of shear reinforcement will allow the designer to compare the benefits of using (or not using) shear reinforcement and to determine which type is most desirable for the given structure. This capability will result in more efficient or effective designs as reflected by lower cost structures without the loss of blast resistant capacity.

Objective

4. The overall objective of this research program is to better understand the effects of shear reinforcement details on slab behavior to improve the state-of-the-art in protective construction design, for both safety and cost effectiveness. The study is directed toward understanding how shear reinforcement details affect the large-deflection behavior of one-way slabs. This is not particularly a study of shear stresses in slabs, but rather a study of the effects of reinforcement normally considered to be shear reinforcement on the large-deflection behavior of slabs.

5. A primary objective is to determine how shear reinforcement details interact with other physical details to affect the response limits of a slab. The work reported herein is directed toward the development of new guidelines for designing shear reinforcement in blast-resistant structures.

Scope

6. A literature search was conducted to gather available test data of reinforced concrete slabs loaded to failure or to large deflections (statically and dynamically). Woodson (Reference 1) conducted a review of tests on one-way slabs and beams containing stirrups. The most difficult task of this phase of the study was the collection of data on slabs containing lacing. The available data was in the form of research papers and technical reports. Of course, different authors address different concepts and details; therefore, not all design parameters were presented in some of the reports.

7. The known design parameters and parameters associated with the structural response of the slabs were tabulated and entered into a Lotus 1-2-3 file for future manipulation. Some discussion of the data is presented in this report. Also, a summary of current design criteria found in the design manuals is presented, and data are compared to the criteria.

8. A brief description of current analytical/design theories based on truss-model analogy is presented. These theories will be the primary bases for the remaining work in this study. In addition, recommendations for new guidelines (response limits) for the design of protective structures to resist the effects of conventional weapons are given.

CHAPTER 2: CURRENT PRACTICE

9. In conventional design the primary source of design guidance for placement of reinforcing steel in reinforced concrete structures, including shear reinforcement, is the American Concrete Institute's ACI 318-83 (Reference 2). No such single, widely accepted criteria document exists for blast resistant design guidance; however, the most widely used reference in the area of designing for explosive safety is the Tri-Service Manual, "Structures to Resist the Effects of Accidental Explosions," (References 3 and 4). Other references for guidance include the Army manual on Protective Construction, TM 5-855-1 (Reference 5) and the NATO Semihardened Design Criteria document published by the U.S. Air Force (Reference 6). A summary of the guidance for shear reinforcement from each of these references follows.

The Tri-Service Manual, "Structures to Resist the Effects of Accidental Explosions":

10. The Tri-Service Manual is the most widely used manual for structural design to resist blast effects. Its Army designation is TM 5-1300, for the Navy it is NAVFAC P397, and for the Air Force it is AFM 88-22. For convenience it will be referred to as TM 5-1300 (Reference 3) in this paper. A recently completed revision of TM 5-1300 is available in draft form and criteria from volume IV of the draft (Reference 4) will also be discussed here.

11. In Section 3-11 of TM 5-1300 (Reference 3), the use of lacing is required for "close-in" detonations, i.e. whenever pressures much larger than 200 psi are expected. The use of unlaced concrete elements is allowed at lower pressures if support rotations of less than 2 degrees are predicted.

12. In volume IV of the new draft version of TM 5-1300 (Reference 4) these restrictions are relaxed slightly. Considering the resistance-deflection relationship for flexural response of a reinforced concrete element, Section 4-9.1 of the manual states that, within the range following yielding of the flexural reinforcement, the compression concrete crushes at a deflection corresponding to 2 degrees support rotation. This crushing of the compression concrete is considered to be "failure" for elements without shear reinforcement. For elements with shear reinforcement (single-leg stirrups or lacing reinforcement) which properly tie the flexural reinforcement, the crushing of the concrete results in a slight loss of capacity since the compressive force is transferred to the compression reinforcement. As the reinforcement enters into its strain-hardening region, the resistance increases with increasing deflection. Section 4-9.1 of the manual states that single-leg stirrups will restrain the compression reinforcement for a short time into its strain hardening region until failure of the element occurs at a support rotation of 4 degrees. It further states that lacing reinforcement will restrain the flexural reinforcement through its entire strain-hardening region until tension failure of the principal reinforcement occurs at a support rotation of 12 degrees. Draft TM 5-1300 distinguishes between a "close-in" design range and a "far" design range for purposes of predicting the mode of response. In the far design range, the distribution of the applied loads is considered to be fairly uniform and deflections required to absorb the loading are comparatively small. Section 4-9.2 states that non-laced elements are considered to be adequate to resist the far-design loads with ductile behavior within the constraints of the allowable support rotations previously discussed. The design of the element to undergo

deflections corresponding to support rotations between 4 and 12 degrees requires the use of laced reinforcement. An exception is when the element has sufficient lateral restraint to develop in-plane forces in the tensile-membrane region of response. In this case, Section 4-9.2 states that the capacity of the element increases with increasing deflection until the reinforcement fails in tension. A value of support rotation is not given here, but one might deduce that a support rotation of 12 degrees is intended since it is the value given in Section 4-9.1 for tension failure of the reinforcement in a laced slab. However, a value of 8 degrees is given elsewhere in the draft manual as a limit of support rotation for elements containing stirrups and experiencing tensile membrane behavior.

13. Section 4-9.3 of the Draft TM 5-1300 discusses ductile behavior in the close-in design range. Again, the maximum deflection of a laced element experiencing flexural response is given as that corresponding to 12 degrees support rotation. This section states the following:

> "Single leg stirrups contribute to the integrity of a protective element in much the same way as lacing, however, the stirrups are less effective at the closer explosive separation distances. The explosive charge must be located further away from an element containing stirrups than a laced element. In addition, the maximum deflection of an element with single leg stirrups is limited to 4 degrees support rotation under flexural action or 8 degrees under tension membrane action. If the charge location permits, and reduced support rotations are required, elements with single leg stirrups may prove more economical than laced elements."

14. Section 4-25.3 of the Draft TM 5-1300 explains that for simplicity, the energy absorbed under the actual resistance-deflection curve with a maximum support rotation of 12 degrees, is approximated with an elastic-plastic model having a maximum support rotation of 8 degrees as shown in Figure 4-18 of the

manual. This figure is reproduced in Figure 2 of this paper. Due to the use of this model, one might presume that the criteria for maximum support rotation is identical for non-laced elements with lateral support and laced elements. However, no such elastic-plastic analogy is given for laced slabs. All other discussion in the draft manual indicates that the 12 degrees support rotation for laced elements is not equivalent to the 12 degrees support rotation for non-laced elements modeled with 8 degrees using the elastic-plastic curve. For example, Section 4-32 states:

> "... Also, the blast capacity of laced elements are greater than corresponding (same concrete thickness and quantity of reinforcement) elements with single leg stirrups. Laced elements may attain deflections corresponding to 12 degrees support rotation whereas elements with single leg stirrups are designed for a maximum rotation of 8 degrees. These non-laced elements must develop tension membrane action in order to develop this large support rotation. If support conditions do not permit tension membrane action, lacing reinforcement must be used to achieve large deflections."

15. It is implied throughout Draft TM 5-1300 that laced elements may attain support rotations of 12 degrees whether they are restrained against lateral movement or not. The manual also implies that a non-laced element may only achieve its maximum support rotation of 8 degrees when it is restrained against lateral movement.

16. In addition to being required for large-deflection behavior, lacing reinforcement is required in slabs subjected to blast at scaled distances less than 1.0 ft/(lbs$^{1/3}$). Section 4-9.4 of the Draft TM 5-1300 indicates that lacing reinforcement is required due to the need to limit the effects of post-failure fragments resulting from flexural failure. It is implied that the size of failed sections of laced elements is fixed by the location of the yield lines, whereas the failure of an unlaced element results in a loss of

structural integrity and fragments in the form of concrete rubble. Section 4-22 discusses the use of single-leg stirrups in slabs at scaled distances between 1.0 and 3.0. Support rotations in slabs with stirrups are limited to 4 degrees in the close-in design range unless support conditions exist to induce tensile membrane behavior. In addition, a non-laced element designed for small deflections in the close-in design range is not reusable and, therefore, cannot sustain multiple incidents.

Army Technical Manual 5-855-1

17. TM 5-855-1 (Reference 5) is intended for use by engineers involved in designing hardened facilities to resist the effects of conventional weapons. The manual includes design criteria for protection against the effects of a penetrating weapon, a contact detonation, or the blast and fragmentation from a standoff detonation.

18. Chapter 9 of TM 5-855-1 discusses the design of shear reinforcement. The criteria presented is primarily based on the guidance of ACI 318-83 (Reference 2) with consideration of available test data. The maximum allowable shear stress to be contributed by the concrete and the shear reinforcement is given as $11.5(f'_c)^{1/2}$ for design purposes as compared to $8(f'_c)^{1/2}$ given by ACI 318-83. An upper bound to the shear capacity of members with web reinforcing is given as that corresponding to a 100 percent increase in the total shear capacity outlined by ACI 318-83 and consisting of contributions from the concrete and shear reinforcing. An important statement concerning shear reinforcement in one-way slabs and beams is given in Section 9-7 and reads as follows:

"Some vertical web reinforcing should be provided for all flexural members subjected to blast loads. A minimum of 50-psi shear stress capacity should be provided by shear steel in the form of stirrups. In those cases where analysis indicates a requirement of vertical shear reinforcing, it should be provided in the form of stirrups."

19. TM 5-855-1 states that shear failures are unlikely in normally constructed two-way slabs, but that the possibility of shear failure increases in some protective construction applications due to high-intensity loads. Shear is given as the governing mode of failure for deep, square, two-way slabs. In the event shear capacity is required above that provided by the concrete alone, additional strength can be provided in the form of vertical and/or horizontal web reinforcing. For beams, one-way slabs, and two-way slabs, the manual recommends a design ductility ratio of 5.0 to 10.0 for flexural design.

USAFE Semihard Design Criteria

20. The purpose of the document (Reference 6) is to give guidance for semihardened and protected facilities with conventional, nuclear, biological, and chemical weapon protection. It states that these structures shall be designed to provide a ductile response to blast loading. Ductility of structural members is considered imperative to provide structural economy, energy absorption capability and to preclude catastrophic (brittle) failures. For design, a ductility ratio of 10 may be used, or theoretical joint rotations should be less than 4 degrees. Designers are to consider allowable degrees of dynamic structural deformation when sizing members and determining steel reinforcement amounts. Where explosive testing provides a sufficient data base, designers may size structural members to duplicate the performance of acceptable specimens in the data base. Structural deformations must not

prohibit function operation of the structure nor produce dangerous, high velocity, concrete spall fragments. All reinforced concrete sections are required to be doubly reinforced (both faces) in both longitudinal and transverse directions. Where flexural response is significant, the structural element is to be reinforced symmetrically, i.e. the compression and tension reinforcement is the same. The use of stirrups is discussed as follows:

> "Ties and/or stirrups shall be provided in all members to provide concrete confinement, shear reinforcement, and to enable the element to reach its ultimate section capacity. Without stirrups, cracking and dislodgment of the concrete from between the reinforcement layers and buckling of the compression steel usually produce failure long before the ultimate strain of the reinforcement and the maximum energy absorption are attained. Stirrups contribute to the integrity of the element in the following ways:
>
> a. The ductility of the primary flexural steel is developed.
>
> b. Integrity of the concrete between the two layers of flexural reinforcement is maintained.
>
> c. Compression reinforcement is restrained from buckling.
>
> d. High shear stresses at the supports are resisted.
>
> e. The resistance to local shear failure produced by the high intensity of the peak blast pressures is increased.
>
> f. Quantity and velocity of post-failure fragments are reduced. Stirrups shall be bent a minimum of 135 degrees around the interior face steel and 90 degrees around the exterior face steel. Shear, splice, and anchorage details shall receive added design attention. Designers shall refer to protective design manuals and/or seismic design manuals for appropriate details."

21. The document does not address the use of laced reinforcement. The above list of ways that stirrups enhance the integrity of structural elements is similar to the wording given in TM 5-1300 for the ways that lacing enhances the integrity of structural elements, except for the stirrup details given in Item f above.

Summary of Design Criteria

22. The above review indicates that guidance documents differ considerably on the type of shear reinforcement required; however, the use of some type of shear reinforcement is uniformly required for blast design. The current TM 5-1300 (Reference 3) limits the use of stirrups to those elements designed to undergo support rotations of less than 2 degrees. The Draft TM 5-1300 (Reference 4) allows the use of stirrups in elements designed to undergo support rotations of up to 8 degrees for scaled ranges greater than one and when restraint against lateral support movement exists. Lacing bars are required by References 3 and 4 for most cases and in every case for "close-in" detonations. Although TM 5-855-1 and the USAFE Semihardened Criteria do not require lacing, they do require some form of shear reinforcement in all elements designed to resist blast loads.

CHAPTER 3: GENERAL DESCRIPTION OF TEST SERIES

23. This chapter presents a general description of the available experimental data. Any static or dynamic test data on reinforced concrete slabs is considered applicable to this study. Data on composite slabs, members consisting of two reinforced concrete slabs separated by a layer of soil, is not considered applicable. In some cases, the design of the specimens and the experimental results are compared with the guidelines of the Draft TM 5-1300 (Reference 4). A more detailed description of the design parameters and structural response of the specific slabs will be presented in Chapter 4; however, the test series identification numbers used in the tables of Chapter 4 are used to organize this discussion and to allow cross-referencing of these two chapters.

K-82 and SB-82 Series

24. Kiger, Eagles, and Baylot (Reference 7) statically tested three one-way slabs and dynamically tested two one-way slabs as part of a study to evaluate the effects of soil cover on the capacity of earth-covered slabs. Two of the statically tested slabs were buried at a depth of L/2 and one was tested at surface flush. The dynamically tested slabs were companions to the buried statically tested slabs. The principal steel ratio was 0.5 percent in each face. A moderate percentage of closed-hoop stirrups was used in each slab. The results showed that the capacity of the slab buried in sand was substantially greater than either the surface-flush slab or the slab buried in clay due to soil arching. Soil arching acted to distribute much of the load from the center region of the slab to the supports.

B-83 Series

25. Baylot and others (Reference 8) conducted three static tests on one-way slab elements as part of a program to investigate the vulnerability of buried structures to conventional weapons. All slabs had the same percentages of steel in the top and bottom and were constructed with single-leg stirrups. Although large supports rotations were not achieved, the tests supported the fact that slabs with adequate lateral support will develop a significant enhancement in ultimate capacity due to compressive membrane action.

W-83 Series

26. Woodson (Reference 1) tested ten one-way reinforced concrete slabs, primarily to investigate the effects of stirrups and stirrup details on the load response behavior of slabs. The slabs were rigidly restrained at the supports and were loaded with uniformly distributed pressure. The slabs had span-to-effective-depth ratios of about 12, and principal reinforcement ratios of about 0.008 in each face. Support rotations between 13 and 21 degrees were observed. Figure 3 is a posttest view of the slabs. Due to the increase in resistance with increasing deflections of a slab with a large number of single-leg stirrups, the loading of the slab was not terminated until support rotations were approximately 21 degrees (see Figure 4). A slab having no shear reinforcement achieved support rotations greater than 16 degrees without failure. These slabs had sufficient lateral restraint to develop in-plane forces in the tensile membrane region of response. In this case, TM 5-1300 (Reference 3) would require lacing for support rotations greater than 2 degrees and the Draft TM 5-1300 (Reference 4) would allow a slab with single-leg stirrups to undergo maximum support rotations up to only 8 degrees. The

slab with 21 degrees of support rotation contained single-leg stirrups (135-degree bend on one end and a 90-degree bend on the other end) spaced at about 0.4 d (d = effective depth of slab). The maximum spacing allowed in the Draft TM 5-1300 is 0.5 d and 180-degree bends are required on each end of the stirrup.

W-84 Series

27. Woodson and Garner (Reference 9) statically tested fifteen one-way slabs to determine the effects of principal steel percentages and details on slab behavior. A posttest view of the slabs is shown in Figure 5. All but two of the slabs had approximately the same total area of continuous longitudinal steel as that of the W-83 series. However, the distribution of the total area of principal steel was varied. Principal steel details which were investigated included the use of dowels at the supports, the use of bent bars, and the use of cut-off bars. A group of slabs with bent bars and closely spaced stirrups were tested to determine the expected scatter in experimental results for slabs with identical construction details. All slabs were rigidly restrained at the supports and loaded with uniformly distributed pressure.

28. The steel details that resulted in the best overall performance were a combination of bent-up and straight principal steel. This combination resulted in 75 percent of the total steel in the tension zone at midspan and at the supports. The single-leg stirrups were spaced at about 0.4 d. Many of the slabs in this series contained no shear reinforcement, and one slab contained only bent-up bars. Nearly all of the slabs sustained support rotations greater than 20 degrees. The failure mode was primarily a 3-hinged mechanism with a compressive membrane enhancement and a load-bearing increase in the tensile membrane region. The best tensile membrane enhancement

occurred in the test in which all principal steel consisted of bent-up bars and no stirrups were used. However, due to the lack of any confining steel, large sections of concrete fell from the slab at the locations of the steel bends. The series demonstrated that principal steel details significantly affect the ductility or large-deflection behavior of a one-way slab.

G-84 Series

29. Guice (Reference 10) statically tested 16 one-way reinforced concrete slabs with uniformly distributed load, primarily to investigate the effects of edge restraint on slab behavior. Each slab contained single-leg stirrups spaced at approximately 1.5 d (compared to a minimum of about 0.5 d required by Reference 4). Again, the stirrups had 135 degree bends on one end and 90 degree bends on the other end. Support rotations of about 20 degrees were sustained. Regardless of support rotational freedom, the tests showed that the percentage of load carried by tensile membrane action is dependent upon the slab's span-to-thickness ratio. Guice concluded that elements which have a span-to-thickness ratio of about 15, have 1.0 to 1.5 percent of steel in each face, and are supported with a relatively large lateral stiffness and a moderate rotational stiffness will probably result in a structure which best combines the characteristics of strength, ductility, and economy.

K4S-69 and K4D-69 Series

30. Keenan (Reference 11) tested four laced reinforced concrete one-way slabs. All slabs were supported at clamped ends and longitudinally restrained. One slab was tested with an increasing static load applied by water pressure, and the other three slabs were subjected to two or more short-duration dynamic loads. Keenan reported that the rotation capacity at the

critical sections of the slab was greater than 9.2 degrees, but could not be measured due to safety limitations on the loading device. Slab behavior was similar under static and dynamic load. The type of loading did not change the extent of cracked or crushed concrete, the collapse mechanism, the mode of failure, or the rotation capacity at supports. Keenan reported that the stress in the lacing bars at the hinges was induced by rotation of the cross-section in addition to shear. Lacing bars yielded at midspan, where the shear is theoretically zero. No lacing bars yielded under static load, but some yielded under dynamic load. The tests showed that the effects of rotation, in addition to shear, should be considered in designing lacing reinforcement for sections near a support.

<u>K9S-69 and K9D-69 Series</u>

31. Keenan (Reference 12) tested nine reinforced concrete two-way slabs. Six slabs were tested under uniform static pressure, and three slabs were tested under dynamic loads of long duration. The slabs were square and restrained against rotation and longitudinal movement at the edges. Keenan discussed the observation of tensile-membrane fragments that were the size of the reinforcing mesh in a slab that contained no lacing at midspan. This slab only had lacing near the supports and contained no stirrups. It was observed that lacing prevented this type of fragmentation in a slab with lacing at midspan. However, lacing did not prevent severe spalling. It was concluded that slabs should contain lacing or closely spaced principal reinforcement to prevent fragmentation caused by dynamic deflections in the tensile membrane region of behavior. None of the slabs contained stirrups.

32. Although the new Draft TM 5-1300 does not address the use of closely

spaced principal reinforcement, test data indicate that using smaller principal reinforcing bars with a reduced spacing will enhance the ductile response of slabs. This is reported by Keenan (References 11 and 12) and Woodson (Reference 1).

K-78,79 and FH-78,79 Series

33. Kiger and Getchell (References 13 through 18) conducted seven dynamic tests and four static tests investigating the effects of load intensity, backfill type, and depth-of-burial on the response of one-way roof slabs of box elements. The dynamic tests were conducted with 1/4-scale box structures loaded by simulated nuclear overpressures utilizing a Foam HEST (High Explosive Simulation Technique). The static tests were conducted on 1/8-scale structures in the Large Blast Load Generator at WES. The slabs had equal percentages of tension and compression steel and contained closely spaced stirrups. All of the structures were tested under soil cover, and the study demonstrated that soil cover helped to redistribute the load on the structure.

34. Figure 6 shows the damage to a box (FH3-78) buried 2 feet deep in clay and subjected to a simulated nuclear overpressure of about 2000 psi peak pressure. Permanent deflection was about 6 inches (about 14 degrees support rotation) with some concrete cover broken free. In another test (FH4-79), a box was buried 10 inches in sand and loaded at about 2000 psi peak pressure. Figure 7 shows a partial failure of the roof and some loss of concrete cover from the reinforcement (Figure 8). Permanent roof deflections were about 12.5 inches (approximately 28 degrees support rotation). Although the roof was clearly on the verge of collapse, it did sustain this level of damage at a very high pressure without catastrophic failure.

S-83, F-83, and F-84 Series

35. Slawson and others (Reference 19) conducted six static and twelve (four were repeated dynamic loads) dynamic tests investigating structural design, structural response in various backfills, the effects of concrete strength on response, and the effects of repeated hits on structural response. Tests were performed on two element types. The Type 1 element (S1-83 and F1-83) was a two-bay box structure with structural steel interior column supports, and the Type 2 element was an open-end box element. The slabs contained single-leg stirrups at a moderate spacing and most of the roof slabs in the static tests sustained support rotation greater than 15 degrees.

FS-1-63 and 1/3-1-63 Series

36. Rindner and Schwartz (Reference 20) summarize tests conducted up through December, 1964, in support of the establishment of design criteria for facilities used for operations dealing with explosives. Eleven dynamic tests were conducted primarily to investigate the validity of scale-model testing. The slabs were tested in a horizontal position, resting on timber supports on the ground. Both full-scale prototypes and one-third scale models were tested. The range of damage extended from surface pitting to complete destruction producing rubble. In most of the tests, the supporting timbers were displaced and badly damaged. Donor charges were placed at various standoff distances and consisted of bare cylinders of Composition B for the smaller charges, but the explosive was encased in 1/8-thick pipe for the larger charges. The tests showed a good qualitative correlation between full-scale and 1/3-scale models under similar loading and support conditions. None of the slabs contained any shear reinforcement and all contained only

about 0.15 percent principal reinforcement in each face. The scaled standoff distances (z) varied from approximately 1.0 to 2.6 ft/lb$^{1/3}$, and damage varied from slight to complete failure and even small rubble.

FS-64 and 1/3-64 Series

37. A second series of scaling investigation tests are summarized in Reference 20. Six slabs (three full-scale and three 1/3-scale) were tested to further investigate the feasibility of one-third scale testing and to investigate different methods of slab support that would allow photographic coverage of slab fragment movement. Four of the slabs were supported by structural steel frames. The supports were destroyed by blasts in the vertical tests of this series. None of the slabs contained shear reinforcement and scaled distances (z) varied from approximately 1.0 to 2.6 ft/lb$^{1/3}$. Slab damage ranged from surface cracking to break-up of the slab into a few sections. The one-third scale slabs displayed brittle failure characteristics while the full-scale slabs tended to crack and deflect.

CAM-64 Series

38. As summarized in Reference 20, these three tests were conducted to further investigate methods of slab support that would allow photographic coverage of slab fragment movement. Two of the slabs were supported in a horizontal position on heavy steel plates on edge. The third slab was supported in a vertical position by walls of a steel tunnel. None of the slabs contained any shear reinforcement, and z values were approximately 0.5 ft/lb$^{1/3}$ in each test. Each slab was completely destroyed.

BAL-64 Series

39. These two slabs (Reference 20) were constructed with balanced steel percentages of approximately 1.3 percent in each face. No shear reinforcement was used. One slab was tested at a z-value of 0.5 and one at a z-value of 2.5. For z = 0.5, the slab was reduced to small rubble. For z = 2.5, the slab experienced heavy damage with large cracks and rubble.

1/3-2-64 and 1/3-S1-64 Series

40. These tests are also summarized in Reference 20 and were conducted to investigate the responses of various basic types of slabs when subjected to different loading conditions. All of the slabs were supported using the steel-walled test tunnel. Two steel percentages were used: 0.15 percent in each face (5 slabs) and 0.40 percent in each face (4 slabs). No shear reinforcement was used. Z-values ranged from approximately 0.5 to 3.5 $ft/lb^{1/3}$. The extent of the damage ranged from hairline cracks to complete destruction.

1/3-65 Series

41. Rindner, Wachtell, and Saffian (Reference 21) summarize tests conducted during 1965 for the establishment of design criteria. Thirty-one tests conducted in that year are applicable to this study. The tests were conducted to:

a. establish the explosive quantity range for specially reinforced concrete

b. establish a general configuration of reinforced concrete (plain, composite, etc.) which will be used in the construction of explosive facilities

c. evaluate the blast loading (impulse) applied to the wall

d. investigate the optimum amount of reinforcement and the maximum amount of reinforcement that is feasible in cubicle construction

e. evaluate specific detailing of reinforcement (various kinds of shear reinforcement, placement of reinforcement).

42. Principal steel percentages varied from 0.44 percent to 2.7 percent. Most of the slabs contained no shear reinforcement, but ten slabs contained lacing. One slab contained "looped" shear reinforcement. Z-values ranged from approximately 0.4 to 1.6. The slabs were either supported in the steel tunnel or in the "new support structure" design for charges over 30 lbs. Bending restraint plates were also used in some of the tests, but these particular slabs were not laterally restrained. It was concluded that a substantial increase in slab capacity is accomplished by strengthening the slab (using a higher percentage of reinforcement) and by the proper use of ties (shear reinforcing) which significantly increased the resistance to blast.

<u>1/3-66 and 1/8-66 Series</u>

43. Rinder, Wachtell, and Saffian (Reference 22) discuss this series conducted in 1966 which included both 1/3-scale and 1/8-scale model slabs. The slabs were either supported in the steel tunnel or the new support structure. Some of the slabs were bolted in the new support structure with one row of bolts at each support. The tests were conducted to:

a. determine both qualitative and quantitative data on slab response

b. investigate the effects of high and low compression strength concrete and the addition of fibrous materials (cut wire and nylon).

c. determine the validity of 1/8-scale testing.

Twenty-eight of the slabs are applicable to this study. Most of the slabs contained lacing. One slab contained looped reinforcement, and six slabs had

no shear reinforcement. Z-values ranged from 0.3 to 1.25 $lb/ft^{1/3}$. Damage levels ranged from slight damage to total destruction.

1/3-67 Series

44. Rinder, Wachtell, and Saffin (Reference 23) summarize the tests conducted during 1967 for the establishment of design criteria. Seventeen slabs of the series are applicable to this study. Principal steel percentages ranged from 0.65 to 2.70 percent. All of the slabs were bolted into the "modified new support structure" which included the use of lateral restraining plates. All of the slabs contained laced reinforcement, and z-values ranged from 0.50 to 1.65. The slabs were tested to obtain data for the design of reinforced concrete laced elements subjected to close-in blasts. The tests also evaluated the use of fibrous reinforced concrete for reducing spall and the use of low compressive strength concrete (2,500-3,000 psi).

45. It was concluded that the impulse capacity of reinforced slabs containing fibers is larger than that of slabs without fibrous material. There was no significant loss in capacity due to the reduced concrete strength. An important conclusion was that incipient failure of a laced reinforced concrete element may be described by a maximum deflection corresponding to a support rotation of 12 degrees.

T-88 Series

46. Tancreto (Reference 24) is currently testing two-way slabs to verify the design criteria for slabs with tensile membrane resistance, and to investigate the effect of stirrup design on the response of reinforced concrete slabs at large support rotations (> 4 degrees) and for close-in explosions. Six of the

proposed dynamic tests have been conducted. Four of the slabs contained stirrups, one contained lacing, and one had no shear reinforcement. The slabs were not loaded to failure. The tests indicated that the breaching criteria is conservative since stirrups were adequate at $z = 0.7$ ft/lb$^{1/3}$ which is less than 1.0. Stirrup spacings of d were adequate as opposed to d/2. Tancreto concluded that more tests than his remaining five tests are needed to establish:

a. improved breaching criteria

b. allowable stirrup spacing (for flexural ductility and for shear)

c. allowable maximum rotation from flexural resistance with stirrups

d. ultimate rotation with tensile membrane resistance.

DS-81 and DS-82 Series

47. Slawson (Reference 25) dynamically tested eleven shallow-buried reinforced concrete box elements, primarily to evaluate dynamic shear failure criteria. The structures were subjected to high-pressure (greater than 2000 psi peak pressure) short-duration loads. Shear reinforcement consisted of single-leg stirrups with a 90-degree bend and a 135-degree bend. When dynamic shear failure occurred, severing the roof slab from the walls, the concrete was severely crushed and fell from the roof slab reinforcement mats when lifted from the floor for post-test examination.

48. The one-way roof slabs of four of Slawson's structures did not experience total collapse. One of these roof slabs, having a span-to-effective-depth ratio of 10, experienced a deflection at midspan of about 10 inches for the 48-inch clear span (about 23 degrees support rotation). Some spalling occurred at the walls, but the rest of the slab was cracked without spalling

action (see Figure 9). This slab contained single-leg stirrups spaced at about 0.8 d with two stirrups at each location. The remaining three slabs contained one single-leg stirrup at each location, and the spacing varied from about 0.25 d near the supports to 0.5 d at midspan. These slabs had span-to-effective-depth ratios of 7. One slab responded predominantly in shear with a permanent midspan deflection of about 4.5 inches. The unloaded face of the slab experienced cracking with disintegration of the concrete occurring only at the supports. Another roof slab experienced a midspan deflection of about 12 inches (about 26 degrees support rotation). The concrete cover spalled, and the concrete between the principal reinforcement mats was broken up over the entire span but did not fall from the reinforcement cage (see Figure 10). These data indicate that slabs with single-leg stirrups can resist high-pressure short-duration loads without total collapse.

1/8-MC-71 Test

49. Levy and others (Reference 26) discuss a test on an 1/8-scale model cubicle wall. The structure contained lacing and 0.4 percent principal steel in each face. A z-value of 0.5 ft/lb$^{1/3}$ was used. The structure successfully withstood the loading with heavy damage but without failure of any reinforcement.

B-84 Series

50. Baylot (Reference 27) dynamically tested a 1/4-scale reinforced concrete model of a weapon storage cubicle using a foam HEST (High Explosive Simulation Technique). Three layers of reinforcement were provided in the principal direction in the long walls, roof, and floor, while two layers were provided in the transverse direction. The three layers consisted of one layer near

the center of the cross section of that element. The shear steel ratio of 0.0031 was provided in the form of single-leg stirrups near the roof slab reports. The stirrups had a 135-degree bend at one end and a 90-degree bend at the other. The L/t ratio of the one-way roof slab was approximately 14.8, and the span length was 79.5 inches.

51. The HEST simulated a 2.5 kiloton weapon with a peak pressure of approximately 1500 psi. The midspan deflection of the roof slab was approximately 11.4 inches, which corresponds to a support rotation of approximately 16-degrees. The first two rows of stirrups along the exterior wall were either broken or straightened out. A very small shallow zone of concrete crushing occurred down the center of the top surface of the roof slab. The largest crack on the bottom surface was approximately 1/8-inch wide.

KW-87 Test

52. A full-scale 100-man capacity blast shelter was tested in a simulated nuclear overpressure environment (Reference 28). The 3-bay structure had a roof span of about 11 feet for each bay, a roof thickness of about 10.25 inches, and average tension and compression steel ratios of 0.011 and 0.0036, respectively. Some principal steel (25 percent) was "draped" so that it served as tensile reinforcement at both the supports (top) and center (bottom) of the roof as in the W-84 series. No shear reinforcement was used in the roof, and the bottom face of the roof was corrugated sheet metal that served as form work and effectively prevented spallation of the concrete from the roof. A posttest view of the interior of Bay 1 is shown in Figure 11. Maximum roof deflection was 17 inches (about 14 degrees support rotation).

Due to the protection of the thin metal covering the roof, no concrete spall can be seen.

F-77 Series

53. Fuehrer and Keeser (Reference 29) conducted a test program to provide data defining the vulnerability of underground concrete targets. The objective was to generate experimental data relating the maximum distances at which explosive charges of specified weights are capable of breaching reinforced concrete slabs with varying span-to-thickness ratios. A total of 23 tests were conducted with charge weights from 4.6 to 27 pounds. Maximum standoff distance at which target slabs were breached increased with decreasing valves of span-to-thickness ratios.

B-85 and H-89 Series

54. Eleven tests were conducted in the B-85 series (Reference 30) to study the response of structures buried in sand to the loading from a point-source detonation. Each test involved a reinforced concrete test slab and a cylindrical-cased charge. The parameters that were varied included the charge orientation, standoff distance, span-to-thickness ratio, and the percentage of reinforcing steel in the test slab.

55. The H-89 series (Reference 30) were conducted as a follow-up to the B-85 series to investigate the effects of backfill type. A breach occurred in the test with the low-shear-strength, low seismic velocity reconstituted clay backfill, and light damage occurred in the test with the high-shear-strength, low seismic velocity sand backfill.

Summary

56. A considerable amount of applicable data is available in the literature. Very little research has been performed on stirrup slabs and lace slabs in the same study. Because of the varied objectives of the test series, some parameters are not known for some of the specimens, and results are often not reported in great detail. Also, many of the specimens were not loaded to failure. However, the data base is useful for the comparison of design parameters and for an indication of the degree of conservatism of current design criteria. Several examples of dynamic and static tests on structures containing stirrups demonstrated that rotations in excess of 20 degrees without failure are possible. Support rotations of over 14 degrees were sustained in one case for both static and dynamic tests on slabs with no shear reinforcement at all.

CHAPTER 4: DATA PRESENTATION AND GENERAL DISCUSSION

57. This chapter presents the known construction parameters and results of the available pertinent tests. All of the tests are part of a series described in Chapter 3. Data for a total of 258 tests are presented. Fifty-four of the tests were static loadings of one-way slabs, and ten were static loadings of box elements. One-hundred, twenty-one of the tests were dynamic loadings of slabs, most of which were one-way slabs. Seventy-three tests were dynamic loadings of the box-type structures. These tests were conducted by the U.S. Army Engineer Waterways Experiment Station (WES), the Air Force Armament Laboratory, the U.S. Naval Civil Engineering Laboratory (NCEL), or the Picatinny Arsenal (PA).

Data Notation

58. The test data are presented in Tables 4.1 through 4.4 and in Figures 13 through 16. An explanation of the notation used in these tables is given in this section. The element identification number is given in the first column of each table and usually begins with the initial of the author of the report on that particular study. The general form of these identification numbers were used in Chapter 3 for cross-referencing of the data with the tests descriptions. The identification number also includes the year that the report or paper for the test was published. The identification number is most useful for the study of the dynamic slab tests data of Table 4.3. In this case, most of the numbers contain four parts that may be described as follows:

A-B-C-D

where

A: FS (full scale); 1/3 (1/3-scale); 1/8 (1/8-scale)

B: 1 (standard slab 1)
2 (standard slab 2)
S1 (strengthened slab 1)
S2 (strengthened slab 2)
etc.

C: year of test series

D: consecutive numbering of specimens

59. The "restraint" column defines the support conditions. Most of the static slab tests were clamped at the supports with steel plates and are considered as rigid. The support structure of the G-84 series allowed some rotational freedom, resulting in partial restraint. The slabs of the box elements were either supported at two or at four sides by walls of the box. Again, this parameter was varied the most for the dynamic slabs tests. The notation is as follows:

- H-1 - Slab in horizontal position and supported on horizontal wood blocks.
- H-2 - Slab in horizontal position and supported on vertical steel blocks.
- H-3 - Slab in horizontal position and supported on horizontal steel blocks.
- H-4 - Slab in horizontal position in steel tunnel.
- V-1 - Slab bolted in modified "new structure" with lateral restraining plates.
- V-2 - Slab in vertical position, located in steel cubicle, and supported by steel frame.
- V-3 - Slab in vertical position and supported by steel tunnel.
- V-4 - Slab in vertical position and supported in steel tunnel or in "new structure".
- V-5 - Slab in vertical position in steel tunnel or "new structure" with bending restraint plates, but not laterally restrained.
- V-6 - Slab in vertical position in "new structure", bolted with one row of bolts at each support.
- V-7 - Slab bolted in modified "new structure" with lateral restraining plates.

60. Most of the dynamic slab tests were conducted by PA. The reports on many of those tests did not present some of the parameters listed in the tables. In particular, the effective depth of the slab (d), the concrete compressive strength (f_c'), the steel yield strength (f_y), the spacing of the principal steel (s), and the spacing of the shear reinforcement (S_s) were often not reported.

61. The thickness of the slab (t) was always reported. Therefore, the clear-span-to-thickness ratio (L/t) is presented in the tables rather than the more commonly used L/d ratio. Similarly the ratios of principal steel spacing to thickness (s/t) and shear reinforcement spacing to thickness (S_s/t) are given where known. The tension steel percentages (p) and compression steel percentages (p') at the midspan and the support are reported for all slabs. The shear reinforcement ratio (p_s) is also known for all slabs.

62. The scaled range or standoff distance (z) in ft/lb$^{1/3}$ is presented for all dynamic tests except for the HEST tests in which z is not appropriate. The type of reinforcing bars used for the principal steel is presented for some of the dynamic tests. It is known that nearly all of the static tests were constructed with heat-treated deformed wire. The notation for the reinforcement type for the dynamic tests is as follows:

RB = commercial reinforcing bar
CWF = commercial welded wire fabric
CWW = commercial welded wire

The distinction is made because reinforcing bar is generally more ductile than deformed wire. The type of reinforcing bar may also affect the bond between the steel and the concrete due to variations in (or lack of) deformations on the surface of the steel.

63. For the static tests and a few of the dynamic tests, the support rotation (Θ) at test termination or collapse is presented. The permanent deflection (Δ_{perm}) is reported for the dynamic tests when known.

64. The general load-deflection curve for a reinforced concrete slab may be described as in Figure 12. The ultimate resistance (u) is defined by point A. The incipient failure load (I) is the load resistance occurring when the structure is about to collapse and loose its load-carrying ability. For a ductile slab experiencing tensile membrane behavior, the incipient failure load is at point C of Figure 12. For a brittle slab, I and u may have nearly the same value. The ratio I/u is presented for the static tests since the load-deflection curve is easily obtained in static tests.

65. The "Remarks" section of the tables includes comments about special construction details and the test results. The definitions of other symbols used in the remarks section as well as in some of the other columns are given below.

DOB - depth of burial
3-H - 3-hinged mechanism
3-HM - 3-hinged mechanism membrane
4-H - 4-hinged mechanism
Θ_R - allowed rotational freedom at supports
b - undetermined
S - shear failure
c - collapse
P_{so} - peak surface overpressure
MD - Medium Damage - less than incipient failure condition, light spalling
HD - Heavy Damage - at or around incipient failure condition, scabbing and/or crushed concrete between reinforcement
PD - Partial Destruction - slab broken-up but remaining in one piece
TD - Total Destruction - slab broken-up completely, producing flying fragments
d - Slab loaded in chamber with explosives distributed in firing tubes

General Discussion

66. All of the statically tested slabs were laterally restrained such that compressive and tensile membrane forces could be developed. However, as noted by Guice (Reference 10), slabs of the G-84 series with large rotational freedom were not able to achieve their potential compressive membrane capacity because of large, early deflections. Therefore, the slab snapped through to the tensile membrane stage before significant thrusts were developed. For the thinner slabs (smaller L/t ratio) of the G-84 series, this snap-through occurred for smaller rotational freedoms than that of the thicker slabs. Small rotational freedoms enhanced the tensile membrane capacity and the incipient collapse deflection of the slabs.

67. The L/t values for all of the statically tested slabs were large enough to ensure that the slabs were not "deep" slabs, and that a flexural response mode could be expected. All of the statically tested slabs had nearly equal percentages of steel in the top and bottom faces except for the W-84 series. In that series, it was found that ductility increased when more of the total area of principal reinforcement was placed in the tension zones. The compressive strength of the concrete for these slabs ranged from about 3.6 to 5 ksi except for the K-82 and B-83 series, where values from 6.1 to 6.9 ksi were reported. The yield strength of the principal steel was also greater for these two series as it ranged from approximately 70 to 90 ksi. Additionally, all but one of the slabs of these two series had principal steel percentages of around 0.5 percent, compared to about 0.75 to 1.6 percent for the other series of slabs. Ignoring the two slabs of the K-82 series with soil cover, the slabs of these two series were similar to the other statically tested slabs for all other parameters; yet, these slabs failed at relatively small

support rotations. In general, a steel with a high yield strength is less ductile than that of Grade 60 or less steel. The presence of stirrups or closed hoops did not alter the fact that a slab with a low percentage of such steel and having a high concrete strength will result in brittle behavior.

68. A close spacing of stirrups was shown to enhance large-deflection behavior in the W-83 series. A S_s/t ratio of about 0.33 (or less than about d/2) was required before stirrup spacing had an effect on the behavior of those slabs.

69. The static slab tests of Table 4.1 demonstrated that slabs with single-leg stirrups (or even no shear reinforcement) can achieve large support rotations without collapse.

70. The static box tests of Table 4.2 were all tested in a buried configuration. Otherwise, values of the construction parameters were in the same general range of the static slabs of Table 4.1. One box (K4-79) had a L/t ratio of only 3.3 and failed in shear without rupture of any reinforcement. Large support rotations were achieved in many of the static box tests, all of which contained single- or double-leg stirrups.

71. All but one (1/8-MC-71) of the dynamic box tests of Table 4.4 were one-way slabs and were part of the same research programs as the static tests. The boxes contained either stirrups or no shear reinforcement, and construction parameters were similar to those of the static tests. Of these dynamic box tests, only element F2-83 was tested surface flush. The other boxes were buried. The 1/8-MC-71 was a two-way slab with lacing and no soil cover. This was also the only box that was not tested in a HEST configuration. The scaled range (z) was 0.5 for this box, and it experienced heavy damage but no reinforcement was ruptured.

72. The largest group of tests is that of the dynamic slab tests presented in Table 4.3. Most of these tests were conducted in the 1960's with the objective of developing design criteria for the original TM 5-1300 (1969). Many other slabs of "composite" construction were tested in the same program, but were not appropriate for this study. A composite slab consists of two slabs with a filler material such as sand placed or "sandwiched" between them. Table 4.3 shows that the slabs contained either laced reinforcement or no shear reinforcement. Only two slabs (1/3-S12-65-1 and 1/3-S12-66-1) contained stirrups or "looped" reinforcement. Therefore, it is not surprising that TM 5-1300 imposes significant limitations on slabs with stirrups - no data was available. Of those two slabs with looped reinforcement, one was tested at $z = 1.25$ and experienced only medium damage with no reinforcement failure (TM 5-1300 requires lacing when $z < 1.0$). The other slab with looped reinforcement was tested at $z = 1.0$ and was described as incurring partial destruction with all tension steel failing and shear failure in the concrete. This slab was not laterally restrained; therefore, tensile membrane forces could not be developed. Also, both of these slabs had a L/t ratio of 6.0. This L/t ratio is approaching that of a deep slab where ductile behavior is less likely to occur for moderately reinforced slabs.

73. Principal steel percentages varied considerably among the dynamic slab tests. Slab 1/3-S14-65-1 contained a high percentage of steel in each face (2.7 percent) but no shear reinforcement. A z-value of 0.5 was used and L/t was equal to 4. The slab experienced only medium damage with all steel intact. A laced slab (1/3-S13-65-1) with the same parameter values except for L/t equal to 6 incurred heavy damage with tension steel failing at the supports and at midspan.

74. Some of the slabs with no shear reinforcement failed in large sections and not small rubble. For example, slab 1/3-1-63-5 was tested at z = 0.99 with L/t = 8 and was broken into 2 large sections. Three of the slabs with no shear reinforcement were tested at z = 0.80. The rest of these slabs were tested at a z of about 1 or greater or a z of about 0.5. The three slabs tested at z = 0.80 had L/t values of 6 and over twice as much compression steel as tension steel at midspan and vice versa at the supports. Based on the W-84 series of static tests, the most effective use of a given total area of principal steel is the placement of more of the steel in the tension zones at both midspan and the supports. These three slabs experienced total destruction.

75. There were laced slabs that also experienced heavy damage. It is obvious that the laced slabs responded better than the slabs with no shear reinforcement, but the limits of slabs without shear reinforcement cannot be determined from these tests. This series makes almost no contribution to the understanding of the behavior of slabs containing stirrups.

76. The T-88 series is the only set of dynamic slab tests which is aimed toward some comparison of laced and stirrup slabs. As discussed in Chapter 3, six of the proposed tests have been conducted. Only one of these six slabs contained lacing, and one contained no shear reinforcement. The slabs were not tested to failure and many parameters were varied, making it difficult to quantify the relative effectiveness of lacing and stirrups. However, the tests thus far have indicated that slabs with stirrups can achieve support rotations greater than those allowed by the Draft TM 5-1300. These were two-way slabs with large L/t ratios of 15 or 20. Tancreto (Reference 24) at NCEL

is conducting these tests and concluded that more research is needed to determine the rotation capacity and tensile membrane behavior of slabs with stirrups, the allowable stirrup spacing, and to improve breaching criteria.

77. A more in-depth discussion of the data is presented in Chapter 5.

TABLE 4.1 STATIC SLAB TESTS

ELEMENT	RESTRAINT	L/t	MIDSPAN p	p'	SUPPORT p	p'	(ksi) fy	(ksi) fc'	d (in)	t (in)	s/t	db (in)	SHEAR REINFORCEMENT	Ps %	Ss/t	Theta	I/U	
K1-82	RIGID	8.3	0.50	0.50	0.50	0.50	90.2	6.7	2.40	2.90	0.69	0.18	closed hoop	0.25	0.69	0.50	1.00	3-W,
K2-82	RIGID	8.3	0.50	0.50	0.50	0.50	90.2	6.8	2.40	2.90	0.69	0.18	closed hoop	0.25	0.69	5.70	0.90	DOB=
K3-82	RIGID	8.3	0.50	0.50	0.50	0.50	90.2	6.9	2.40	2.90	0.69	0.18	closed hoop	0.25	0.69	12.9	0.32	DOB=
B1-83	RIGID	10	0.47	0.47	0.47	0.47	77.7	6.1	1.95	2.40	0.83	0.15	135-S-135	0.23	0.81	5.20	0.77	3-W
B2-83	RIGID	10	1.04	1.04	1.04	1.04	70.1	6.1	1.88	2.40	0.69	0.21	135-S-135	0.98	0.46	3.30	1.00	3-W,
B3-83	RIGID	5.0	0.46	0.46	0.46	0.46	70.1	6.1	4.30	4.80	0.69	0.21	135-S-135	0.41	0.35	3.10	1.00	3-W,
W1-83	RIGID	10.4	0.74	0.74	0.85	0.85	59.8	4.8	1.94	2.31	1.62	0.25	NONE	--	--	16.3	0.72	3-W,
W2-83	RIGID	10.4	0.74	0.74	0.85	0.85	59.8	4.9	1.94	2.31	1.62	0.25	135-S-135	0.36	0.33	20.6	1.02	3-WM
W3-83	RIGID	10.4	0.74	0.74	0.85	0.85	59.8	5.1	1.94	2.31	1.62	0.25	135-S-135	0.18	0.65	14.0	0.63	3-W,
W4-83	RIGID	10.4	0.74	0.74	0.85	0.85	59.8	4.9	1.94	2.31	1.62	0.25	135-S-135	0.09	1.30	13.1	0.55	3-W,
W5-83	RIGID	10.4	0.74	0.74	0.85	0.85	59.8	5.1	1.94	2.31	1.62	0.25	135-S-135	0.18	0.65	15.4	0.88	Temp
W6-83	RIGID	10.4	0.74	0.74	0.85	0.85	59.8	4.9	1.94	2.31	1.62	0.25	135-S-90	0.18	0.65	14.0	0.72	3-W,
W7-83	RIGID	10.4	0.74	0.74	0.85	0.85	59.8	5.0	1.94	2.31	1.62	0.25	135-S-90	0.18	0.65	14.5	0.85	Temp.
W8-83	RIGID	10.4	0.74	0.74	0.85	0.85	59.8	5.1	1.94	2.31	1.62	0.25	Double 135	0.18	0.65	14.0	0.78	3-W,
W9-83	RIGID	10.4	0.75	0.75	0.86	0.86	62.4	4.7	1.94	2.31	0.76	0.18	135-S-135	0.19	0.65	16.3	0.79	3-W,
W10-83	RIGID	10.4	0.75	0.75	0.86	0.86	62.4	4.9	1.94	2.31	0.76	0.18	135-S-135	0.38	0.33	18.4	1.12	3-W,
W1-84	RIGID	10.4	0.74	0.74	0.85	0.85	66.0	4.5	1.94	2.31	1.62	0.25	NONE	--	--	18.4	0.73	3-W,
W2-84	RIGID	10.4	0.79	0.79	0.79	0.79	66.0	4.5	1.81	2.31	1.62	0.25	NONE	--	--	19.7	0.85	3-W,
W3-84	RIGID	10.4	1.14	0.40	0.40	1.14	63.5	4.5	1.81	2.31	1.62	0.30	NONE	--	--	21.0	0.85	3-W,
W4-84	RIGID	10.4	1.14	0.40	1.19	1.14	63.5	4.5	1.81	2.31	1.62	0.178 0.30	NONE	--	--	20.6	0.85	#2 d *
W5-84	RIGID	10.4	1.14	0.40	1.19	1.14	63.5	4.5	1.81	2.31	1.62	0.178 0.30	NONE	--	--	23.4	0.68	#2 d *
W6-84	RIGID	10.4	1.58	0.00	1.58	0.00	66.0	4.5	1.81	2.31	1.62	0.25	NONE	--	--	14.0	1.47	#2 p
W7-84	RIGID	10.4	1.13	0.45	1.13	0.45	66.0	4.3	1.81	2.31	1.62	0.25	NONE	--	--	19.7	0.91	Alter
W8-84	RIGID	10.4	1.13	0.45	1.13	0.45	66.0	4.3	1.81	2.31	1.62	0.25	135-S-90	0.06	1.30	23.4	0.93	Alter
W9-84	RIGID	10.4	1.13	0.45	1.13	0.45	66.0	4.0	1.81	2.31	1.62	0.25	135-S-90	0.22	0.32	23.4	1.04	Alter
W10-84	RIGID	10.4	1.13	0.45	1.13	0.45	66.0	4.0	1.81	2.31	1.62	0.25	135-S-90	0.22	0.32	23.4	0.85	Alter
W11-84	RIGID	10.4	1.13	0.45	1.13	0.45	66.0	4.2	1.81	2.31	1.62	0.25	135-S-90	0.22	0.32	19.3	0.74	Alter
W12-84	RIGID	10.4	1.13	0.45	1.13	0.45	66.0	4.2	1.81	2.31	1.62	0.25	135-S-90	0.22	0.32	24.6	0.99	Alt #
W13-84	RIGID	10.4	1.13	0.45	1.13	0.45	66.0	4.2	1.81	2.31	1.62	0.25	NONE	--	--	22.6	0.70	Alter
W14-84	RIGID	8.3	1.02	1.02	1.02	1.02	60.3	3.6	2.40	2.90	0.69	0.25	135-S-90	1.53	0.55	22.6	0.76	3-W,
W15-84	RIGID	10.4	0.79	0.45	0.79	0.45	66.0	3.6	1.81	2.31	1.62	0.25	NONE	--	--	24.2	0.69	3-W,
G1-84	PARTIAL	10.4	0.52	0.52	0.52	0.52	50.0	4.4	1.94	2.31	1.30	0.20	135-S-90	0.22	1.30	17.1	0.44	Theta
G2-84	PARTIAL	10.4	0.52	0.52	0.52	0.52	50.0	4.3	1.94	2.31	1.30	0.20	135-S-90	0.22	1.30	19.3	0.65	Theta
G3-84	PARTIAL	10.4	0.74	0.74	0.74	0.74	58.5	4.4	1.94	2.31	1.62	0.25	135-S-90	0.18	1.30	18.9	1.13	Theta
G4-84	PARTIAL	10.4	0.74	0.74	0.74	0.74	58.5	4.3	1.94	2.31	1.62	0.25	135-S-90	0.18	1.30	18.0	1.07	Theta
G4A-84	PARTIAL	10.4	0.74	0.74	0.74	0.74	58.5	4.2	1.94	2.31	1.62	0.25	135-S-90	0.18	1.30	20.1	1.38	Theta
G4B-84	PARTIAL	10.4	0.74	0.74	0.74	0.74	58.5	4.2	1.94	2.31	1.62	0.25	135-S-90	0.18	1.30	12.7	0.86	Theta
G5-84	PARTIAL	10.4	1.06	1.06	1.06	1.06	58.5	4.4	1.94	2.31	1.08	0.25	135-S-90	0.27	1.30	16.3	0.85	Theta
G6-84	PARTIAL	10.4	1.06	1.06	1.06	1.06	58.5	4.3	1.94	2.31	1.08	0.25	135-S-90	0.27	1.30	16.3	1.33	Theta
G7-84	PARTIAL	14.8	0.58	0.58	0.58	0.58	67.3	5.0	1.25	1.63	2.31	0.18	135-S-90	0.18	1.85	17.1	0.84	Theta
G8-84	PARTIAL	14.8	0.58	0.58	0.58	0.58	67.3	5.0	1.25	1.63	2.31	0.18	135-S-90	0.18	1.85	15.8	1.00	Theta
G9-84	PARTIAL	14.8	1.14	1.14	1.14	1.14	58.5	5.0	1.25	1.63	2.31	0.25	135-S-90	0.18	1.85	16.7	2.23	Theta
G9A-84	PARTIAL	14.8	1.14	1.14	1.14	1.14	58.5	5.0	1.25	1.63	2.31	0.25	135-S-90	0.18	1.85	18.0	2.24	Theta
G10-84	PARTIAL	14.8	1.14	1.14	1.14	1.14	58.5	5.0	1.25	1.63	2.31	0.25	135-S-90	0.18	1.85	16.7	LARGE	Theta
G10A-84	PARTIAL	14.8	1.14	1.14	1.14	1.14	58.5	5.0	1.25	1.63	2.31	0.25	135-S-90	0.18	1.85	11.3	LARGE	Theta
G11-84	PARTIAL	14.8	1.47	1.47	1.47	1.47	58.5	5.0	1.25	1.63	1.69	0.25	135-S-90	0.24	1.85	14.9	2.52	Theta
G12-84	PARTIAL	14.8	1.47	1.47	1.47	1.47	58.5	5.0	1.25	1.63	1.69	0.25	135-S-90	0.24	1.85	14.5	4.45	Theta
K4S-69	RIGID	12	2.11	2.11	2.11	2.11	49.9	5.0	4.875	6.0	0.25	0.63	LACE	1.37	0.25	9.2	1.25	Test t
K9S1-69	RIGID 2-WAY	24	0.82	0.82	0.82	0.82	49.6	3.6	2.25	3.0	2.0	0.38	LACE	0.19	0.5	8.7	0.90	Loaded
K9S2-69	RIGID 2-WAY	24	0	0	0	0	49.6	4.1	-	3.0	-	-	NONE	0	-	1.6	1.00	Loaded
K9S3-69	RIGID 2-WAY	24	0.82	0.82	0.82	0.82	49.6	4.1	2.25	3.0	2.0	0.38	LACE	0.19	0.5	13.3	1.23	Loaded
K9S4-69	RIGID 2-WAY	24	0.82	0.82	0.82	0.82	49.6	3.3	2.25	3.0	2.0	0.38	LACE	0.19	0.5	12.6	0.95	Loaded
K9S5-69	RIGID 2-WAY	15.2	0.89	0.89	0.89	0.89	47.4	3.2	3.75	4.75	1.26	0.5	LACE	0.42	0.42	10.8	0.87	Loaded
K9S6-69	RIGID 2-WAY	12	1.33	1.33	1.33	1.33	47.4	3.6	5.00	6.0	0.5	0.5	LACE	1.67	0.17	1.8	0.79	Loaded

t (in)	s/t	db (in)	SHEAR REINFORCEMENT	Ps %	Ss/t	Theta	I/U	REMARKS
2.90	0.69	0.18	closed hoop	0.25	0.69	0.50	1.00	3-H, test term. @ U, 100% tension steel rupt. @ midspan
2.90	0.69	0.18	closed hoop	0.25	0.69	5.70	0.90	DOB=L/2, 3-H, 100% tension steel rupt. @ midspan
2.90	0.69	0.18	closed hoop	0.25	0.69	12.9	0.32	DOB=L/2, 3-H, 100% tension and 50% comp. steel rupt. @ midspan
2.40	0.83	0.15	135-S-135	0.23	0.81	5.20	0.77	3-H
2.40	0.69	0.21	135-S-135	0.98	0.46	3.30	1.00	3-H, test term. @ U
4.80	0.69	0.21	135-S-135	0.41	0.35	3.10	1.00	3-H, test term. @ U
2.31	1.62	0.25	NONE	--	--	16.3	0.72	3-H, 86% tension steel rupt. @ midspan, 50% tension steel rupt. @ support
2.31	1.62	0.25	135-S-135	0.36	0.33	20.6	1.02	3-HM, [illegible]0% ten. & 43% comp. steel rupt. @ midspan, 64% ten. rupt. @ support.
2.31	1.62	0.25	135-S-135	0.18	0.65	14.0	0.63	3-H, 1[illegible]0% tension rupt. @ midspan, 43% tension rupt. @ support
2.31	1.62	0.25	135-S-135	0.09	1.30	13.1	0.55	3[illegible], 100% tension rupture @ midspan, 29% tension rupture @ support.
2.31	1.62	0.25	135-S-135	0.18	0.65	15.4	0.88	Temp. steel outside, 3-H, 86% ten. rupt. @ midspan, 14% ten. rupt. @ support.
2.31	1.62	0.25	135-S-90	0.18	0.65	14.0	0.72	3-H, 71% tension rupture @ midspan, 14% tension rupture @ support.
2.31	1.62	0.25	135-S-90	0.18	0.65	14.5	0.85	Temp. steel outside, 3-H, 86% ten. rupt. @ midspan, 14% ten. rupt. @ support.
2.31	1.62	0.25	Double 135	0.18	0.65	14.0	0.78	3-H, 71% tension rupture @ midspan, 14% tension rupture @ support.
2.31	0.76	0.18	135-S-135	0.19	0.65	16.3	0.79	3-H, 100% tension rupture @ midspan, 39% tension rupture @ support.
2.31	0.76	0.18	135-S-135	0.38	0.33	18.4	1.12	3-H, 100% tension & 57% comp rupture @ midspan, 71% tension rupt. @ support.
2.31	1.62	0.25	NONE	--	--	18.4	0.73	3-H, 100% tension @ midspan & 7% tension @ support ruptured.
2.31	1.62	0.25	NONE	--	--	19.7	0.85	3-H, 100% tension @ midspan & 14% tension @ support ruptured.
2.31	1.62	0.30	NONE	--	--	21.0	0.85	3-H, 71% tension @ midspan & 86% tension @ support ruptured.
2.31	1.62	0.178 0.30	NONE	--	--	20.6	0.85	#2 dowels @ supports. 3-HM, 43% ten. @ midspan & 7% ten. @ support ruptured. *
2.31	1.62	0.178 0.30	NONE	--	--	23.4	0.68	#2 dowels @ supp. 3-HM, 71% ten. & 29% comp rupt. @ midspan, 14% ten. @ supp. *
2.31	1.62	0.25	NONE	--	--	14.0	1.47	#2 pairs bent. 4-H, no steel ruptured.
2.31	1.62	0.25	NONE	--	--	19.7	0.91	Alternate #2 pairs bent. 3-HM, 40% ten. @ midspan & 10% ten. @ supp ruptured.
2.31	1.62	0.25	135-S-90	0.06	1.30	23.4	0.93	Alternate #2 pairs bent. 3-HM, 60% ten. @ midspan & 20% ten. @ supp. rupt.
2.31	1.62	0.25	135-S-90	0.22	0.32	23.4	1.04	Alternate #2 pairs bent. 3-HM, 80% ten. @ midspan & 20% ten. @ supp. rupt.
2.31	1.62	0.25	135-S-90	0.22	0.32	23.4	0.85	Alternate #2 pairs bent. 3-HM, 60% ten & 25% comp @ mid & 45% ten @ supp rupt.
2.31	1.62	0.25	135-S-90	0.22	0.32	19.3	0.74	Alternate #2 pairs bent. 3-H, 100% ten & 50% comp @ mid & 45% ten @ supp rupt.
2.31	1.62	0.25	135-S-90	0.22	0.32	24.6	0.99	Alt #2 pairs bent. 3-HM, temp steel out., 50% ten @ mid & 25% ten @ supp rupt.
2.31	1.62	0.25	NONE	--	--	22.6	0.70	Alternate #2 pairs cut. 3-HM, 40% tension @ midspan & 15% ten @ supp rupt.
2.90	0.69	0.25	135-S-90	1.53	0.55	22.6	0.76	3-H, 100% tension @ midspan & 100% tension @ support rupture.
2.31	1.62	0.25	NONE	--	--	24.2	0.69	3-H, 100% tension @ midspan & 57% tension @ support rupture.
2.31	1.30	0.20	135-S-90	0.22	1.30	17.1	0.44	Theta R = 1.82, 3-H, 100% tens & 100% comp @ midspan & 88% tens @ supp rupt.
2.31	1.30	0.20	135-S-90	0.22	1.30	19.3	0.65	Theta R = 1.56, 3-H, 100% tens & 88% comp @ midspan & 88% ten @ supp rupt.
2.31	1.62	0.25	135-S-90	0.18	1.30	18.9	1.13	Theta R = 1.24, 3-HM, 71% tension @ midspan & 29% tension @ support rupture.
2.31	1.62	0.25	135-S-90	0.18	1.30	18.0	1.07	Theta R = 1.50, 3-HM, 43% tension @ midspan & 14% tension @ support rupture.
2.31	1.62	0.25	135-S-90	0.18	1.30	20.1	1.38	Theta R = 2.52, 3-HM, 57% tension @ midspan rupture.
2.31	1.62	0.25	135-S-90	0.18	1.30	12.7	0.86	Theta R = 2.20, 3-HM, 29% tension @ midspan rupture.
2.31	1.08	0.25	135-S-90	0.27	1.30	16.3	0.85	Theta R = 0.55, 3-HM, 30% tension @ midspan & 40% tension @ support rupture.
2.31	1.08	0.25	135-S-90	0.27	1.30	16.3	1.33	Theta R = 2.04, 3-HM, 20% tension @ midspan rupture.
1.63	2.31	0.18	135-S-90	0.18	1.85	17.1	0.84	Theta R = 0.61, 3-H, 86% tension & comp @ midspan & 93% ten @ supp rupt.
1.63	2.31	0.18	135-S-90	0.18	1.85	15.8	1.00	Theta R = 2.20, 3-H, 100% ten & 86% comp @ midspan & 93% ten @ supp rupt.
1.63	2.31	0.25	135-S-90	0.18	1.85	16.7	2.23	Theta R = 1.29, 3-HM, 14% tension @ support rupture.
1.63	2.31	0.25	135-S-90	0.18	1.85	18.0	2.24	Theta R = 0.40, 3-HM, 14% tension @ support rupture.
1.63	2.31	0.25	135-S-90	0.18	1.85	16.7	LARGE	Theta R = 2.79, pure tensile membrane, 3-HM, no steel rupture.
1.63	2.31	0.25	135-S-90	0.18	1.85	11.3	LARGE	Theta R = 2.04, pure tensile membrane, 3-HM, 57% tension @ midspan rupture.
1.63	1.69	0.25	135-S-90	0.24	1.85	14.9	2.52	Theta R = 0.76, 3-HM, no steel rupture.
1.63	1.69	0.25	135-S-90	0.24	1.85	14.5	4.45	Theta R = 2.04, 3-HM, no steel rupture.
6.0	0.25	0.63	LACE	1.37	0.25	9.2	1.25	Test terminated due to loading device. 3-H, No steel rupture.
3.0	2.0	0.38	LACE	0.19	0.5	8.7	0.90	Loaded until rupture of steel or water seal, 3-HM.
3.0	-	-	NONE	0	-	1.6	1.00	Loaded until rupture of steel or water seal, 3-H.
3.0	2.0	0.38	LACE	0.19	0.5	13.3	1.23	Loaded until rupture of steel or water seal, 3-HM.
3.0	2.0	0.38	LACE	0.19	0.5	12.6	0.95	Loaded until rupture of steel or water seal, 3-HM.
4.75	1.26	0.5	LACE	0.42	0.42	10.8	0.87	Loaded until rupture of steel or water seal, 3-HM.
6.0	0.5	0.5	LACE	1.67	0.17	1.8	0.79	Loaded until rupture of steel or water seal, 3-H.

TABLE 4.2 STATIC BOX TEST

ELEMENT	RESTRAINT	L/t	MIDSPAN p	MIDSPAN p'	SUPPORT p	SUPPORT p'	(ksi) fy	(ksi) f'c	d (in)	t (in)	s/t	db (in)	SHEAR REINFORCEMENT	Ps %	Ss/t		I/U	
K1-78	4 sides	8.3	1.00	1.00	1.00	1.00	60.0	5.2	2.40	2.9	0.69	0.25	135-S-90	1.53	0.55	5.7	1.00	DOB=L/:
K2-78	4 sides	8.3	1.00	1.00	1.00	1.00	72.0	6.2	2.40	2.9	0.69	0.25	135-S-90	1.53	0.55	3.6	1.00	DOB=L/:
K3-79	4 sides	8.3	1.00	1.00	1.00	1.00	60.0	5.8	2.40	2.9	0.69	0.25	135-S-90	1.53	0.55	4.8	1.00	DOB-L/:
K4-79	4 sides	3.3	1.85	1.85	1.85	1.85	68.0	6.1	6.40	7.3	0.69	0.75	135-S-90	1.10	0.69	9.9	0.91	DOB=L/:
S1-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	6.2	1.94	2.5	1.50	0.25	0-135	0.18	0.60	1.7	1.00	DOB=4L/
S2-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.2	1.94	2.5	1.50	0.25	0-135	0.18	0.60	16.9	0.47	3-H, :
S3-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.2	1.94	2.5	1.50	0.25	0-135	0.18	0.60	15.6	0.46	DOB=4L/
S4-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.6	1.94	2.5	1.50	0.25	0-1[illegible]5	0.18	0.60	7.9	0.72	DOB=4L/
S5-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	3.5	1.94	2.5	1.50	0.25	0-135	0.18	0.60	15.3	0.55	DOB=4L/
S6-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	4.5	1.94	2.5	1.50	0.25	0-135	0.18	0.60	15.3	0.98	DOB=4L/

s/t	db (in)	SHEAR REINFORCEMENT	Ps %	Ss/t		I/U	REMARKS
0.69	0.25	135-S-90	1.53	0.55	5.7	1.00	DOB=L/2. Collapse @ U=I. 100% tension & comp. steel rupt. @ midspan rupt
0.69	0.25	135-S-90	1.53	0.55	3.6	1.00	DOB=L/2. 3-HM. test term. @ U. 80% ten. & 60% comp. @ mid. 100% ten. @ supp
0.69	0.25	135-S-90	1.53	0.55	4.8	1.00	DOB-L/5. collapse @ U=I. 100% tension & comp. steel rupture @ midspan rupt.
0.69	0.75	135-S-90	1.10	0.69	9.9	0.91	DOB=L/5. shear failure. no steel ruptured
1.50	0.25	0-135	0.18	0.50	1.7	1.00	DOB=4L/11. collapse @ U=I. interior support failed
1.50	0.25	0-135	0.18	0.60	16.9	0.47	3-H. 100% tension @ midspan and support rupture
1.50	0.25	0-135	0.18	0.60	15.6	0.46	DOB=4L/11. 3-H. 100% tension @ midspan and support rupture
1.50	0.25	0-135	0.18	0.60	7.9	0.72	DOB=4L/11. 3-H. 100% tension @ midspan and support rupture
1.50	0.25	0-135	0.18	0.60	15.3	0.55	DOB=4L/11. 3-H. 100% tension @ midspan and support rupture
1.50	0.25	0-135	0.18	0.60	15.3	0.98	DOB=4L/11. 3-H. 100% tension @ midspan and support rupture

TABLE 4.3 DYNAMIC SLAB TESTS

ELEMENT	RESTRAINT	L/t	MIDSPAN p	MIDSPAN p'	SUPPORT p	SUPPORT p'	fy	fc'	d (in)	t (in)	s/t	db (in)	SHEAR REINFORCEMENT	Ps %		REINF. TYPE	(FT/LB^1 Z
FS-1-63-1	H-1	8	0.15	0.15	0.15	0.15		4.0		12	1.0	0.50	NONE	--	--	RB	2.62
FS-1-63-2	H-1	8	0.15	0.15	0.15	0.15		4.0		12	1.0	0.50	NONE	--	--	RB	1.68
FS-1-63-3	H-1	8	0.15	0.15	0.15	0.15		4.0		12	1.0	0.50	NONE	--	--	RB	1.08
FS-1-63-4	H-1	8	0.15	0.15	0.15	0.15		4.0		12	1.0	0.50	NONE	--	--	RB	1.04
FS-1-63-5	H-1	8	0.15	0.15	0.15	0.15		4.0		12	1.0	0.50	NONE	--	--	RB	1.67
1/3-1-63-1	H-1	8	0.15	0.15	0.15	0.15		6.0		4	1.0	0.16	NONE	--	--	CHH	1.02
1/3-1-63-2	H-1	8	0.15	0.15	0.15	0.15		6.0		4	1.0	0.16	NONE	--	--	CHH	1.72
1/3-1-63-3	H-1	8	0.15	0.15	0.15	0.15		6.0		4	1.0	0.16	NONE	--	--	CHH	1.01
1/3-1-63-4	H-1	8	0.15	0.15	0.15	0.15		6.0		4	1.0	0.16	NONE	--	--	CHH	1.72
1/3-1-63-5	H-1	8	0.15	0.15	0.15	0.15		6.0		4	1.0	0.16	NONE	--	--	CHH	0.99
1/3-1-63-6	H-1	8	0.15	0.15	0.15	0.15		6.0		4	1.0	0.16	NONE	--	--	CHH	2.59
FS-1-64-1	H-1	8	0.15	0.15	0.15	0.15		>=2.5		12	1.0	0.50	NONE	--	--	RB	2.57
FS-1-64-2	V-1	8	0.15	0.15	0.15	0.15		>=2.5		12	1.0	0.50	NONE	--	--	RB	1.01
FS-1-64-3	V-2	8	0.15	0.15	0.15	0.15		>=2.5		12	1.0	0.50	NONE	--	--	RB	1.01
1/3-1-64-1	V-2	8	0.15	0.15	0.15	0.15		>=5.0		4	1.0	0.16	NONE	--	--	CHF	1.02
1/3-1-64-2	V-1	8	0.15	0.15	0.15	0.15		>=5.0		4	1.0	0.16	NONE	--	--	CHF	1.02
1/3-1-64-3	H-1	8	0.15	0.15	0.15	0.15		>=5.0		4	1.0	0.16	NONE	--	--	CHF	2.57
CAM-1-64-1	H-2	8	0.15	0.15	0.15	0.15		5.1		4	1.0	0.16	NONE	--	--	CHF	0.49
CAM-1-64-2	H-2	8	0.15	0.15	0.15	0.15		5.1		4	1.0	0.16	NONE	--	--	CHF	0.46
CAM-1-64-3	V-3	8	0.15	0.15	0.15	0.15		5.1		4	1.0	0.16	NONE	--	--	CHF	0.47
BAL-64-1	V-3	8	1.30	1.30	1.30	1.30				4	0.6	0.38	NONE	--	--	RB	2.47
BAL-64-2	V-3	8	1.30	1.30	1.30	1.30				4	0.6	0.38	NONE	--	--	RB	0.50
1/3-2-64-1	V-3	8	0.15	0.15	0.15	0.15		4.9		4	1.0	0.16	NONE	--	--	CHH	1.99
1/3-2-64-2	V-3	8	0.15	0.15	0.15	0.15		4.9		4	1.0	0.16	NONE	--	--	CHH	1.51
1/3-2-64-3	V-3	8	0.15	0.15	0.15	0.15		4.9		4	1.0	0.16	NONE	--	--	CHH	0.50
1/3-2-64-4	V-3	8	0.15	0.15	0.15	0.15		4.9		4	1.0	0.16	NONE	--	--	CHH	3.51
1/3-2-64-5	V-3	0	0.15	0.15	0.15	0.15		4.9		4	1.0	0.16	NONE	--	--	CHH	2.52
1/3-S1-64-1	V-3	14	0.40	0.40	0.40	0.40		>=6.0		4	0.50	0.19	NONE	--	--	CHH	0.50
1/3-S1-64-2	V-3	14	0.40	0.40	0.40	0.40		>=6.0		4	0.50	0.19	NONE	--	--	CHH	1.51
1/3-S1-64-3	V-3	14	0.40	0.40	0.40	0.40		>=6.0		4	0.50	0.19	NONE	--	--	CHH	3.51
1/3-S1-64-4	V-3	14	0.40	0.40	0.40	0.40		>=6.0		4	0.50	0.19	NONE	--	--	CHH	1.50
1/3-S2-65-1	V-4	6	0.44	0.44	0.44	0.44				4			NONE	--	--	wire	0.50
1/3-S2-65-2	V-4	6	0.44	0.44	0.44	0.44				4			NONE	--	--	wire	1.25
1/3-S3-65-1	V-4	6	0.65	0.65	0.65	0.65				4			NONE	--	--	RB	0.50
1/3-S3-65-2	V-4	6	0.65	0.65	0.65	0.65				4			NONE	--	--	RB	1.25
1/3-S4-65-1	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	0.50
1/3-S4-65-2	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	0.50
1/3-S4-65-3	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	1.25
1/3-S4-65-4	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	1.00
1/3-S4-65-5	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	0.50
1/3-S4-65-6	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	1.60
1/3-S4-65-7	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	0.55
1/3-S4-65-8	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	0.80
1/3-S4-65-9	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	1.25
1/3-S4-65-10	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	0.80
1/3-S4-65-11	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	0.80
1/3-S4-65-12	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	1.25
1/3-S4-65-13	V-4	6	0.65	1.40	1.40	0.65				4			NONE	--	--	RB	1.00
1/3-S6-65-1	V-4	6	0.65	1.40	1.40	0.65				4			LACE	0.40		RB	0.50
1/3-S6-65-2	V-4	6	0.65	1.40	1.40	0.65				4			LACE	0.40		RB	1.25
1/3-S7-65-1	V-4	2	0.15	0.15	0.15	0.15				12			NONE	--	--	RB	0.50
1/3-S8-65-1	V-4	2	0.69	1.33	1.33	0.69				12			LACE	0.53		RB	0.50
1/3-S9-65-1	V-4	1.85	0.65	1.27	1.27	0.65				13			LACE	0.53		RB	0.40
1/3-S10-65-1	V-5	6	0.65	1.40	1.40	0.65				4			LACE	0.40		RB	1.00
1/3-S10-65-2	V-5	6	0.65	1.40	1.40	0.65				4			LACE	0.40		RB	0.80
1/3-S11-65-1	V-5	6	0.65	0.65	0.65	0.65				4			LACE	0.15		RB	0.80
1/3-S11-65-2	V-5	6	0.65	0.65	0.65	0.65				4			LACE	0.15		RB	1.00
1/3-S12-65-1	V-5	6	0.65	0.65	0.65	0.65				4			LOOP	0.30		RB	1.00
1/3-S13-65-1	V-5	6	2.70	2.70	2.70	2.70				4			LACE	1.20		RB	0.50
1/3-S13-65-2	V-5	6	2.70	2.70	2.70	2.70				4			LACE	1.20		RB	0.50
1/3-S14-65-1	V-5	4	2.70	2.70	2.70	2.70				6			NONE	--	--	RB	0.50
1/3-S15-65-1	V-5	6	0.75	0.75	0.75	0.75				4			NONE	--	--	RB	0.42
1/3-S11-66-1	V-6	6	0.65	0.65	0.65	0.65				4			LACE	0.15		RB	1.00
1/3-S11-66-2	V-6	6	0.65	0.65	0.65	0.65				4			LACE	0.15		RB	1.00
1/3-S11-66-3	V-6	6	0.65	0.65	0.65	0.65				4			LACE	0.15		RB	1.00
1/3-S11-66-4	V-6	6	0.65	0.65	0.65	0.65	Nylon fiber added.			4			LACE	0.15		RB	1.00
1/3-S11-66-5	V-6	6	0.65	0.65	0.65	0.65		Low		4			LACE	0.15		RB	1.00

t (in)	s/t	db (in)	SHEAR REINFORCEMENT	Ps %		REINF. TYPE	(FT/LB^1/3) Z	REMARKS
12	1.0	0.50	NONE	--	--	RB	2.62	Surface pitted. Slight damage.
12	1.0	0.50	NONE	--	--	RB	1.68	Surface pitted, hairline cracks. Slight damage.
12	1.0	0.50	NONE	--	--	RB	1.08	Partial surface crushing, large cracks. Medium damage.
12	1.0	0.50	NONE	--	--	RB	1.04	Partial crushing, small rubble. Complete failure.
12	1.0	0.50	NONE	--	--	RB	1.67	Partial crushing, small rubble. Complete failure.
4	1.0	0.16	NONE	--	--	CHW	1.02	Broken into two sections. Failure
4	1.0	0.16	NONE	--	--	CHW	1.72	Hairline cracks. Slight damage.
4	1.0	0.16	NONE	--	--	CHW	1.01	Reduced to small rubble. Complete failure.
4	1.0	0.16	NONE	--	--	CHW	1.72	Several large sections and small rubble. Complete failure.
4	1.0	0.16	NONE	--	--	CHW	0.99	Broken into two sections. Failure
4	1.0	0.16	NONE	--	--	CHW	2.59	Partial crushing, small rubble. Failure
12	1.0	0.50	NONE	--	--	RB	2.57	Partial crushing, small rubble. Complete failure.
12	1.0	0.50	NONE	--	--	RB	1.01	Medium cracks. Slab displaced 20-30 ft. Slight damage
12	1.0	0.50	NONE	--	--	RB	1.01	Pitted and cracked. Slab and support displaced. Medium damage.
4	1.0	0.16	NONE	--	--	CHF	1.02	Broken into two sections. Failure
4	1.0	0.16	NONE	--	--	CHF	1.02	Broken into two sections. Failure
4	1.0	0.16	NONE	--	--	CHF	2.57	No damage, slight.
4	1.0	0.16	NONE	--	--	CHF	0.49	Reduced to small rubble. Complete failure. (1/3-scale)
4	1.0	0.16	NONE	--	--	CHF	0.46	Reduced to small rubble. Complete failure. (1/3-scale)
4	1.0	0.16	NONE	--	--	CHF	0.47	Reduced to small rubble. Complete failure. (1/3-scale)
4	0.6	0.38	NONE	--	--	RB	2.47	Pitted, large cracks with rubble. Heavy damage. (1/3-scale)
4	0.6	0.38	NONE	--	--	RB	0.50	Reduced to small rubble. Complete failure. (1/3-scale)
4	1.0	0.16	NONE	--	--	CHW	1.99	Broken into two sections. Failure.
4	1.0	0.16	NONE	--	--	CHW	1.51	Broken into 2 sections with supplementary cracks. Failure.
4	1.0	0.16	NONE	--	--	CHW	0.50	Large and small rubble. Complete failure.
4	1.0	0.16	NONE	--	--	CHW	3.51	No wood blocks. Ten steel failed. Small defl. Heavy damage.
4	1.0	0.16	NONE	--	--	CHW	2.52	Large cracks. Medium damage.
4	0.50	0.19	NONE	--	--	CHW	0.50	Center reduced to small rubble. Complete failure.
4	0.50	0.19	NONE	--	--	CHW	1.51	Tension steel failed. Large deflections. Heavy damage.
4	0.50	0.19	NONE	--	--	CHW	3.51	Hairline cracks. Slight damage.
4	0.50	0.19	NONE	--	--	CHW	1.50	Hairline cracks. Slight damage.
4			NONE	--	--	wire	0.50	Total destr. Disintegration at center. Diag failure.
4			NONE	--	--	wire	1.25	Hvy dang. No steel failure. Several major cracks. Spalling.
4			NONE	--	--	RB	0.50	Heavy damage. No steel failure. Bent into two sections.
4			NONE	--	--	RB	1.25	Hvy dang. No steel failure. Several major cracks. Spalling.
4			NONE	--	--	RB	0.50	Hvy dang. No steel failure. Not quite bent into 2 sections.
4			NONE	--	--	RB	0.50	Total destr. Disint. of con. Diag. failure. (+) steel rupt.
4			NONE	--	--	RB	1.25	Total destruction. Disintegration of concrete.
4			NONE	--	--	RB	1.00	Total destruction. Disintegration of concrete.
4			NONE	--	--	RB	0.50	Total destruction. Disintegration of concrete.
4			NONE	--	--	RB	1.60	Partial destruction. Shear failure of concrete.
4			NONE	--	--	RB	0.55	Total destruction. Disintegration of concrete and steel.
4			NONE	--	--	RB	0.80	Total destruction. Disintegration of concrete and steel.
4			NONE	--	--	RB	1.25	Total destruction. Disintegration of concrete.
4			NONE	--	--	RB	0.80	Total destruction. Disintegration of concrete and steel.
4			NONE	--	--	RB	0.80	Total destruction. Disintegration of concrete.
4			NONE	--	--	RB	1.25	Partial destruction. Shear failure.
4			NONE	--	--	RB	1.00	Total destruction. Disintegration of concrete.
4			LACE	0.40		RB	0.50	Partial destr. (+) steel failed. Conc crushed at center.
4			LACE	0.40		RB	1.25	Medium damage. Steel intact. Minor spalling.
12			NONE	--	--	RB	0.50	Total destr. Diag failure. Steel failure. Center shattered.
12			LACE	0.53		RB	0.50	Hvy dang. Steel int. Several cracks tot depth. Deep spall.
13			LACE	0.53		RB	0.40	Reinforcement intact. Heavy spalling.
4			LACE	0.40		RB	1.00	Med damage. Steel intact. Some spalling of both surfaces.
4			LACE	0.40		RB	0.80	Med damage. Steel intact. Some spalling of both surfaces.
4			LACE	0.15		RB	0.80	Partial destruction. Ten. steel failed. Complete spalling.
4			LACE	0.15		RB	1.00	Heavy damage. Failure. of ten. steel @ one supp and center. Spalling 75% both surfaces.
4			LOOP	0.30		RB	1.00	Partial destr. All ten steel failed. Shear failure in conc.
4			LACE	1.20		RB	0.50	Hvy dang. No steel fail. Large defl. Compl spall both side.
4			LACE	1.20		RB	0.50	Hvy dam. Ten steel fail @ both supp & con. 11" defl. @ con.
6			NONE	--	--	RB	0.50	Med dam. All steel intact. Compl spalling of acc. surface.
4			NONE	--	--	RB	0.42	Total destruction. Most steel failed.
4			LACE	0.15		RB	1.00	Heavy damage. All steel intact. Heavy scabbing both faces. Concrete crushed on bottom.
4			LACE	0.15		RB	1.00	Med dam. Wood supp blocks. Steel intact. Acc. spall @ cont. Delta max = 2.5".
4			LACE	0.15		RB	1.00	Hvy dang. Incip. Ten steel failed at cont. Heavy spall. Concrete crushed between steel.
4			LACE	0.15		RB	1.00	Ten. steel failed both supports and center. Conc undamaged. Partial destruction.
4			LACE	0.15		RB	1.00	Total destruction. All steel failed. Conc dislocated from between steel.

TABLE 4.3 DYNAMIC SLAB TESTS (CON'T)

ELEMENT	RESTRAINT	L/t	MIDSPAN p	MIDSPAN p'	SUPPORT p	SUPPORT p'	fy	fc'	d (in)	t (in)	S/t	db (in)	SHEAR REINFORCEMENT	ps %	Ss/t	REINF. TYPE	(ft/lb^1. Z
1/3-S11-66-6	V-6	6	0.65	0.65	0.65	0.65	Cut steel wire added			4			LACE	0.15		RB	1.25
1/3-S12-66-1	V-6	6	0.65	0.65	0.65	0.65				4			LOOP	0.30		RB	1.25
1/3-S13-66-1	V-6	6	2.7	2.7	2.7	2.7	Cut steel wire added			4			LACE	1.2		RB	0.75
1/3-S13-66-2	V-6	6	2.70	2.70	2.70	2.70	nylon fiber added			4			LACE	1.20		RB	0.75
1/3-S13-66-3	V-6	6	2.70	2.70	2.70	2.70	cut steel wire added			4			LACE	1.20		RB	0.75
1/3-S13-66-4	V-6	6	2.70	2.70	2.70	2.70	low			4			LACE	1.20		RB	0.75
1/3-S14-66-1	V-6	4	2.70	2.70	2.70	2.70				6			LACE	1.20		RB	0.50
1/3-S16-66-1	V-6	2	2.70	2.70	2.70	2.70				12			LACE	1.20		RB	0.30
1/8-1-66-1	H-4	6	0.15	0.15	0.15	0.15				1.50			NONE	--			0.46
1/8-S1-66-1	V-3	6	0.65	0.65	0.65	0.65				1.50			LACE	0.15			0.50
1/8-S2-66-1	V-3	6	0.65	1.40	1.40	0.65				1.50			NONE	--			0.50
1/8-S2-66-2	V-3	6	0.65	1.40	1.40	0.65				1.50			NONE	--			0.50
1/8-S2-66-3	V-3	6	0.65	1.40	1.40	0.65				1.50			NONE	--			0.50
1/8-S2-66-4	V-3	6	0.65	1.40	1.40	0.65				1.50			NONE	--			0.50
1/8-S2-66-5	H-4	6	0.65	1.40	1.40	0.65				1.50			NONE	--			0.50
1/8-S3-66-1	V-3	6	0.65	1.40	1.40	0.65				1.50			LACE	0.40			0.50
1/8-S3-66-2	V-3	6	0.65	1.40	1.40	0.65				1.50			LACE	0.40			0.80
1/8-S4-66-1	V-3	6	0.65	1.40	1.40	0.65				1.50			LACE	0.40			0.50
1/8-S4-66-2	V-3	6	0.65	1.40	1.40	0.65				1.50			LACE	0.40			0.40
1/8-S4-66-3	V-3	6	0.65	1.40	1.40	0.65				1.50			LACE	0.40			0.40
1/8-S5-66-1	V-3	4	2.70	2.70	2.70	2.70				2.25			LACE	1.20			0.50
1/8-S5-66-2	V-3	4	2.70	2.70	2.70	2.70				2.25			LACE	1.20			0.50
1/8-S5-66-3	V-3	4	2.70	2.70	2.70	2.70				2.25			LACE	1.20			0.50
1/3-S11-67-1	V-7	6	0.65	0.65	0.65	0.65	low			4			LACE	0.15			1.50
1/3-S11-67-2	V-7	6	0.65	0.65	0.65	0.65	cut steel wire added			4			LACE	0.15			1.50
1/3-S11-67-3	V-7	6	0.65	0.65	0.65	0.65	cut steel wire added			4			LACE	0.15			1.50
1/3-S11-67-4	V-7	6	0.65	0.65	0.65	0.65	nylon fiber			4			LACE	0.15			1.65
1/3-S11-67-5	V-7	6	0.65	0.65	0.65	0.65	nylon fiber			4			LACE	0.15			1.65
1/3-S13-67-1	V-7	6	2.70	2.70	2.70	2.70				4			LACE	1.20			1.00
1/3-S13-67-2	V-7	6	2.70	2.70	2.70	2.70	cut steel wire added			4			LACE	1.20			0.90
1/3-S13-67-3	V-7	6	2.70	2.70	2.70	2.70	cut steel wire added			4			LACE	1.20			0.90
1/3-S13-67-4	V-7	6	2.70	2.70	2.70	2.70	low			4			LACE	1.20			1.00
1/3-S13-67-5	V-7	6	2.70	2.70	2.70	2.70	low			4			LACE	1.20			1.00
1/3-S14-67-1	V-7	4	2.70	2.70	2.70	2.70				6			LACE	1.20			0.50
1/3-S14-67-2	V-7	4	2.70	2.70	2.70	2.70				6			LACE	1.20			0.50

n)	S/t	db (in)	SHEAR REINFORCEMENT	ρs %	Ss/t	REINF. TYPE	(ft/lb^1/3) Z	REMARKS
			LACE	0.15		RB	1.25	Partial destruction. Ten. steel failed. Partial spalling on acceptor surface.
			LOOP	0.30		RB	1.25	No reinforcement failure. Major cracks at corners. Spalling at center. MD.
			LACE	1.2		RB	0.75	Complete surface spalling. Lower 5" of conc disintegrated.
			LACE	1.20		RB	0.75	No spall. Slight cracking on donor side. All steel intact. Delta max = 2 7/8". MD.
			LACE	1.20		RB	0.75	Center of panel crushed. Major cracks @ supp (don.). Spalling in middle (acc). All steel intact. Delta max = 3 1/8". MD.
			LACE	1.20		RB	0.75	All flex steel intact. Several ties failed. Complete spalling (donor and acceptor). MD.
			LACE	1.20		RB	0.50	No steel failure. Complete spalling on both sides. Scabbing at midspan. MD.
			LACE	1.20		RB	0.30	All flex steel intact. Ties fail @ bonds. Slab disinteg. Broken cone fell out. PD.
0			NONE	--			0.46	PD. Broke thru @ center. Large and small fragments. Acceptor face cracked.
0			LACE	0.15			0.50	Don. spalled and cracked @ supp. Acc. spalling. and cracking. Positive steel failure at center.
0			NONE	--			0.50	TD. Pos steel failed @ supports. Center portion of slab completely destroyed.
0			NONE	--			0.50	TD. Pos steel failed @ center. Center portion of slab completely destroyed.
0			NONE	--			0.50	TD. Pos steel failed @ supports. Center portion of slab completely destroyed.
0			NONE	--			0.50	TD. Pos steel failed @ center. Center portion of slab completely destroyed.
0			NONE	--			0.50	TD. Center portion completely destr. Steel broke @ supports (donor) and center (acceptor).
0			LACE	0.40			0.50	PD. Donor badly cracked and broken thru. Acc. broken thru. Pos steel failed.
0			LACE	0.40			0.80	MD. No steel failed. Donor cracking and spalling @ supports. Acceptor cracking and spalling. Delta = 1/4".
0			LACE	0.40			0.50	MD. Donor slightly spalled. Acceptor deeply spalled. No steel failed. No deflection.
0			LACE	0.40			0.40	MD. Donor spalled and cracked. Acceptor deeply spalled. Delta max = 3/16".
0			LACE	0.40			0.40	MD. Donor spalled and cracked. Acceptor deeply spalled. Delta max = 3/16".
5			LACE	1.20			0.50	MD. Donor cracked and spalled. Acceptor deeply spalled. Small deflection.
5			LACE	1.20			0.50	MD. Donor spalled and cracked. Acceptor deeply spalled. Delta max = 3/4".
5			LACE	1.20			0.50	MD. Donor spalled and cracked. Acceptor deeply spalled. Delta max = 3/4".
			LACE	0.15			1.50	MD. Donor (complete spall, one lacing failed @ support, all flex steel intact). Acceptor (complete spalling, all steel intact, Delta=6", horz move. of slab).
			LACE	0.15			1.50	MD. Don. (no spall, flex steel failed in lower 1/2 @ rt. supp. Delta max = 4"). Acceptor (no spall, flex steel failed @ center, just beyond incipient failure.)
			LACE	0.15			1.50	MD. Don. (no spall, flex steel failed in lower 1/2 @ rt. supp. Delta max = 4"). Acceptor (no spall, flex steel failed @ center, just beyond incipient failure).
			LACE	0.15			1.65	MD. No steel fail, no spall, crack @ both supp, comp crush donor center, Delta = 2.9".
			LACE	0.15			1.65	MD. No steel fail, no spall, crack @ both supp, comp crush donor center, Delta = 2.9".
			LACE	1.20			1.00	MD. Don (compl spall exc 8" ver strip, all steel intact, Delta max = 5" horizontal movement). Acc (compl spalling except 6" wide vert. strip @ rt. support, all steel intact).
			LACE	1.20			0.90	MD. Don (no spall, slt crush, all steel intact). Acc (no spall, all steel intact), Delta max = 3 1/4".
			LACE	1.20			0.90	MD. Donor (no spall, slight crushing, all steel intact). Acc. (No spall, all steel intact), Delta max = 3 1/4".
			LACE	1.20			1.00	MD. Complete spalling, all steel intact, Delta max = 3 1/2", horizontal movement).
			LACE	1.20			1.00	MD. Complete spalling, all steel intact, Delta max = 3 1/2", horizontal movement.
			LACE	1.20			0.50	MD. Donor (compl spall, 2 laces broke along left supp, concrete chopped at bottom. Acceptor (complete spall, all steel intact Delta max = 3").
			LACE	1.20			0.50	MD. Don (nearly compl spall, one lace failed @ rt. supp, flex steel intact. Acceptor (compl spall, 3 laces fail @ center.

TABLE 4.3 DYNAMIC SLAB TESTS (CON'T)

ELEMENT	RESTRAINT	L/t	MIDSPAN ρ	MIDSPAN ρ'	SUPPORT ρ	SUPPORT ρ'	fy	fc'	d (in)	t (in)	S/t	db (in)	SHEAR REINFORCEMENT	ρs %	Ss/t	REINF. TYPE	(ft/1… Z
1/3-S16-67-1	V-7	2	2.70	2.70	2.70	2.70				12			LACE	1.20			0.[illegible]
1/3-S16-67-2	V-7	2	2.70	2.70	2.70	2.70				12			LACE	1.20			0.[illegible]
1/3-S17-67-1	V-7	6	2.70	2.70	2.70	2.70				4			LACE	1.20			0.[illegible]
1/3-S17-67-2	V-7	6	2.70	2.70	2.70	2.70				4			LACE	1.20			1.[illegible]
1/3-S18-67-1	V-7	4	2.70	2.70	2.70	2.70				6			LACE	1.20			0.[illegible]
1/3-S18-67-2	V-7	4	2.70	2.70	2.70	2.70				6			LACE	1.20			0.[illegible]
1/3-S18-67-3	V-7	4	2.70	2.70	2.70	2.70				6			LACE	1.20			0.[illegible]
T-1-88	4 sides 2-way slab	20	1.00	1.00	1.00	1.00	74.5	4.0		4.5	0.33	0.25	single 180	0.45	0.33	RB	0.6
T-2-88	4 sides 2-way slab	20	1.00	1.00	1.00	1.00	74.5	4.0		4.5	0.33	0.25	LACE	0.22	0.67	RB	0.7
T-3-88	4 sides 2-way slab	20	1.50	1.50	1.50	1.50	74.5	4.0		4.5	0.56	0.25	single 180	0.48	0.56	RB	0.6
T-4-88	4 sides 2-way slab	20	1.00	1.00	1.00	1.00	74.5	4.0		4.5	0.33	0.25	single 180	0.47	0.67	RB	0.6
T-5-88	4 sides 2-way slab	15	0.31	0.31	0.31	0.31	74.5	4.0		6.0	0.67	0.25	NONE	—	—	RB	1.1
T-6-88	4 sides 2-way slab	20	2.50	2.50	2.50	2.50	66.0	4.0		4.5	0.33	0.38	single 180	0.89 0.45	0.33	RB	0.6
K401-69	Rigid	12	2.11	2.11	2.11	2.11	49.9	5.7	4.88	6.0	0.5	0.63	Lace	1.37	0.5	RB	d
K402-69	Rigid	12	2.11	2.11	2.11	2.11	49.9	5.4	4.88	6.0	0.5	0.63	Lace	1.37	0.5	RB	d
K403-69	Rigid	12	2.11	2.11	2.11	2.11	49.9	5.5	4.88	6.0	0.5	0.63	Lace	1.37	0.5	RB	d
K901-69	Rigid 2-way slab	24	0.82	0.82	0.82	0.82	49.6	3.8	2.25	3.0	2.0	0.38	Lace	0.19	0.5	RB	d
K902-69	Rigid 2-way slab	15.2	0.89	0.89	0.89	0.89	47.4	3.3	3.75	4.75	1.26	0.5	Lace	0.42	0.42	RB	d
K903-69	Rigid 2-way slab	15.2	0.89	0.89	0.89	0.89	47.4	3.6	3.75	4.75	1.26	0.5	Lace	0.42	0.42	RB	d

TS (CON'T)

d (in)	t (in)	S/t	db (in)	SHEAR REINFORCEMENT	Ps %	Ss/t	REINF. TYPE	(ft/lb^1/3) Z	REMARKS
	12			LACE	1.20			0.30	HO. Complete spall both sides. lace fail @ upper 1/2 of slab. no flex steel failure. Delta = 2".
	12			LACE	1.20			0.35	HO. Complete spall both sides. one flex and 2 laces failed @ acceptor center. one lace failed @ donor center. Delta = 2.4".
	4			LACE	1.20			0.90	HO. Don (compl spall except 6" vert strip. all reinf. intact). Acc (comp spalling. all steel intact). Delta max=5". hor mov.
	4			LACE	1.20			1.00	HO. Donor (near complete spall. all steel intact). Acc. (near complete spall all steel intact. Delta max = 5". horz mov.).
	6			LACE	1.20			0.50	HO. Complete spalling. no steel failed. Delta = 1.7"
	6			LACE	1.20			0.50	HO. Complete spalling. no steel failed. Delta = 1.7"
	6			LACE	1.20			0.50	HO. Compl spall. all flex steel intact. one lace failed @ acc. right support. Delta = 3.5".
	4.5	0.33	0.25	single 180	0.45	0.33	RB	0.69	Theta = 10.1. no steel failed.
	4.5	0.33	0.25	LACE	0.22	0.67	RB	0.74	Theta = 9.3. no steel failed.
	4.5	0.56	0.25	single 180	0.48	0.56	RB	0.65	Theta = 10.5. no steel failed.
	4.5	0.33	0.25	single 180	0.47	0.67	RB	0.69	Theta = 12.2. no steel failed.
	6.0	0.67	0.25	NONE	—	—	RB	1.10	Theta = 10.4. no steel fail; 2.5" long shear crack @ 1 supp.
	4.5	0.33	0.38	single 180	0.89 0.45	0.33	RB	0.65	Ps = 0.45 @ center; Theta = 4.8. no steel failed.
4.88	6.0	0.5	0.63	Lace	1.37	0.5	RB	d	Theta=5.2 on 4th loading. Pso = 186 psi. 3-H. No steel rupt.
4.88	6.0	0.5	0.63	Lace	1.37	0.5	RB	d	Theta=9.2 on 2nd loading Pso = 206 psi. 3-H. No steel rupt.
4.88	6.0	0.5	0.63	Lace	1.37	0.5	RB	d	Theta=7.6 on 2nd loading. Pso = 229 psi. 3-H. No steel rupt.
2.25	3.0	2.0	0.38	Lace	0.19	0.5	RB	d	Theta = 0.14. Pso = 10.5 psi. Failed on next cycle. Fragment loose from mesh.
3.75	4.75	1.26	0.5	Lace	0.42	0.42	RB	d	Theta = 1.2. Pso = 87 psi. Destroyed on next loading.
3.75	4.75	1.26	0.5	Lace	0.42	0.42	RB	d	Theta = 1.52. Pso = 91 psi. Destroyed on next loading.

TABLE 4.4 DYNAMIC BOX TESTS

ELEMENT	RESTRAINT	L/t	MIDSPAN p	MIDSPAN p'	SUPPORT p	SUPPORT p'	(ksi) fy	(ksi) fc'	d (in)	t (in)	S/t	db (in)	SHEAR REINFORCEMENT	Ps %	Ss/t	THETA
1/8-MC-71	4 sides	10.0	0.42	0.42	0.42	0.42	91.5	5.7		3.00	0.42	0.14	LACE			
F1-77	2 sides	12	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	0.47
F2-77	2 sides	12	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	1.4
F3-77	2 sides	12	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	7.1
F4-77	2 sides	12	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	2.4
F5-77	2 sides	12	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	c
F6-77	2 sides	12	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	0.7
F7-77	2 sides	12	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	15.2
F8-77	2 sides	9	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	1.0
F9-77	2 sides	9	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	10.4
F10-77	2 sides	9	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	0
F11-77	2 sides	9	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	29.1
F12-77	2 sides	9	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	1.6
F13-77	2 sides	9	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	29.1
F14-77	2 sides	6	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	0
F15-77	2 sides	6	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	7.1
F16-77	2 sides	6	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	26.6
F17-77	2 sides	6	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	c
F18-77	2 sides	18	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	1.8
F19-77	2 sides	18	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	10.2
F20-77	2 sides	18	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	c
F21-77	4 sides	18	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	0
F22-77	4 sides	18	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	2.2
F23-77	4 sides	18	2.0	2.0	2.0	2.0	60	6		4.0	1.0	0.5	NONE	--	--	c
FH1-78	4 sides	8.6	1.00	1.00	1.00	1.00	75.0	7.0	4.80	5.60	0.71	0.50	double	1.50	0.71	1.20
FH2-78.	4 sides	8.6	1.00	1.00	1.00	1.00	57.0	7.6	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	c
FH3-78	4 sides	8.6	1.00	1.00	1.00	1.00	57.0	7.8	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	14.00
FH4-79	4 sides	8.6	1.00	1.00	1.00	1.00	65.0	6.7	4.80	5.60	0.71	1.13	135-S-90	1.50	0.71	26.60
FH5-79	4 sides	8.6	1.50	1.50	1.50	1.50	69.0	6.1	12.00	13.50	0.41	0.50	135-S-90	1.50	0.41	7.50
FH6-79	4 sides	8.6	1.00	1.00	1.00	1.00	65.0	6.8	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	c
FH7-79	3-bay	8.6	1.00	1.00	1.00	1.00	71.0	5.1	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	c
DS1-81	2 sides	8.6	1.00	1.00	1.00	1.00	63.0	3.9	4.80	5.60	0.71	0.50	double	1.50	0.71	c
DS2-81	2 sides	8.6	1.00	1.00	1.00	1.00	63.0	3.9	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	c
DS3-81	2 sides	8.6	1.00	1.00	1.00	1.00	63.0	4.0	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	22.60
DS4-81	2 sides	8.6	1.00	1.00	1.00	1.00	63.0	5.9	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	c
DS5-81	2 sides	8.6	1.00	1.00	1.00	1.00	63.0	6.0	4.80	5.60	0.71	0.50	135-S-90	1.50	0.71	c
DS1-82	2 sides	6.2	0.75	0.75	0.75	0.75	80.0	7.0	6.40	7.30	0.55	0.50	135-S-90	0.50	0.22	c
DS2-82	2 sides	6.2	0.75	0.75	0.75	0.75	80.0	7.7	6.40	7.30	0.55	0.50	135-S-90	0.50	0.22	c
DS3-82	2 sides	6.2	0.75	0.75	0.75	0.75	80.0	7.5	6.40	7.30	0.55	0.50	135-S-90	0.50	0.22	10.40
DS4-82	2 sides	6.2	1.20	1.20	1.20	1.20	67.0	7.4	6.40	7.30	0.55	0.63	135-S-90	0.50	0.22	c
DS5-82	2 sides	6.2	1.20	1.20	1.20	1.20	67.0	7.8	6.40	7.30	0.55	0.63	135-S-90	0.50	0.22	28.20
DS6-82	2 sides	6.2	1.20	1.20	1.20	1.20	67.0	7.3	6.40	7.30	0.55	0.63	135-S-90	0.50	0.22	8.90
SB1-82	rigid	8.3	0.50	0.50	0.50	0.50	90.2	6.9	2.40	2.90	0.69	0.18	closed hoop	0.25	0.69	b
SB2-82	rigid	8.3	0.50	0.50	0.50	0.50	90.2	6.9	2.40	2.90	0.69	0.18	closed hoop	0.25	0.69	3.60
F1-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	6.2	1.94	2.50	1.50	0.25	double 135	0.18	0.60	0.90
F2-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.3	1.94	2.50	1.50	0.25	double 135	0.18	0.60	c
F3-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.0	1.94	2.50	1.50	0.25	double 135	0.18	0.60	0.20
F4-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.0	1.94	2.50	1.50	0.25	double 135	0.18	0.60	1.70
F5-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.1	1.94	2.50	1.50	0.25	double 135	0.18	0.60	3.10
F6-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	3.2	1.94	2.50	1.50	0.25	double 135	0.18	0.60	2.40
F7-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	4.1	1.94	2.50	1.50	0.25	double 135	0.18	0.60	2.30
F8-83	2 sides	13.2	0.69	0.69	0.69	0.69	68.5	5.3	1.94	2.50	1.50	0.25	double 135	0.18	0.60	1.70
F1-84	2 sides	14.7	1.20	0.40	1.6	1.14	63.5	3.1	1.59	2.25	1.00	0.20	--	--	--	1.40
F2-84	2 sides	14.7	1.20	0.40	1.6	1.14	63.5	3.2	1.59	2.25	1.08	0.20	--	--	--	15.30
F3-84	2 sides	14.7	1.20	0.40	1.6	1.14	63.5	3.0	1.59	2.25	1.08	0.20	--	--	--	29.90
F4-84	2 sides	14.7	1.20	0.40	1.6	1.14	63.5	3.3	1.59	2.25	1.08	0.20	--	--	--	0.90
B-84	2 sides	14.8	0.51	0.51	0.51	0.51		4.3	5.38	6.0	0.67	0.18	135-S-90	0.31	0.67	16
B4-85	2 sides	10	0.5	0.5	0.5	0.5	74.4	5.6	3.57	4.3	0.69	0.25	135-S-135	0.32	0.59	1.01
B5-85	2 sides	10	0.5	0.5	0.5	0.5	74.4	6.4	3.57	4.3	0.69	0.25	135-S-135	0.32	0.59	5.66
B5A-85	2 sides	10	0.5	0.5	0.5	0.5	74.4	6.1	3.57	4.3	0.69	0.25	135-S-135	0.32	0.59	c
B6-85	2 sides	10	1.0	1.0	1.0	1.0	63.4	6.4	3.41	4.3	0.69	0.38	135-S-135	0.50	0.59	4.15
B6A-85	2 sides	10	1.0	1.0	1.0	1.0	63.4	6.4	3.41	4.3	0.69	0.38	135-S-135	0.50	0.59	9.58
B7-85	2 sides	5	0.5	0.5	0.5	0.5	63.4	5.7	7.74	8.6	0.35	0.38	135-S-135	0.27	0.35	0.83
B7A-85	2 sides	5	0.5	0.5	0.5	0.5	63.4	5.7	7.74	8.6	0.35	0.38	135-S-135	0.27	0.35	4.65
B8-85	2 sides	10	0.5	0.5	0.5	0.5	74.4	5.6	3.57	4.3	0.69	0.25	135-S-135	0.32	0.59	2.66
B8A-85	2 sides	10	0.5	0.5	0.5	0.5	74.4	5.6	3.57	4.3	0.69	0.25	135-S-135	0.32	0.59	2.50
B9-85	2 sides	10	0.5	0.5	0.5	0.5	74.4	6.0	3.57	4.3	0.69	0.25	135-S-135	0.32	0.59	6.04
B10-85	2 sides	10	0.5	0.5	0.5	0.5	74.4	6.0	3.57	4.3	0.69	0.25	135-S-135	0.32	0.59	4.49
KW-87	4 sides one-way	12.9	1.10	0.36	1.10	0.36	61.6	3.0	7.40	10.30	0.50	0.75	NONE	--	--	14.00
H1-89	2 sides	10	1.0	1.0	1.0	1.0	67.4	6.1	3.41	4.3	0.7	0.38	135-S-135	0.50	0.70	c
H2-89	2 sides	5	0.5	0.5	0.5	0.5	67.4	6.4	7.74	8.6	0.35	0.38	135-S-135	0.28	0.28	2.1
H3-89	2 sides	10	1.0	1.0	1.0	1.0	67.4	5.9	3.41	4.3	0.7	0.38	135-S-135	0.50	0.70	3.8
H4-89	2 sides	10	1.0	1.0	1.0	1.0	67.4	5.9	3.41	4.3	0.7	0.38	135-S-135	0.50	0.70	26.4

t (in)	S/t	db (in)	SHEAR REINFORCEMENT	Ps %	Ss/t	THETA	delta perm	REMARK
3.00	0.42	0.14	LACE					Wall, z = 0.50, no steel failed, heavy damage.
4.0	1.0	0.5	NONE	--	--	0.47	0.2	Z=2.4, open-end box, buried wall, sand, C-4 cylindrical charge, no damage
4.0	1.0	0.5	NONE	--	--	1.4	0.6	Z=1.8, open-end box, buried wall, sand, C-4, small cracks
4.0	1.0	0.5	NONE	--	--	7.1	3.0	Z=1.2, open-end box, buried wall, sand, C-4, major damage, near breach
4.0	1.0	0.5	NONE	--	--	2.4	1.0	Z=2.0, open-end box, buried wall, sand, C-4, small cracks
4.0	1.0	0.5	NONE	--	--	c	c	Z=1.5, open-end box, buried wall, sand, C-4, breach
4.0	1.0	0.5	NONE	--	--	0.7	0.3	Z=2.3, open-end box, buried wall, sand, C-4, small cracks
4.0	1.0	0.5	NONE	--	--	15.2	6.5	Z=1.9, open-end box, buried wall, sand, C-4, breach
4.0	1.0	0.5	NONE	--	--	1.0	0.3	Z=1.8, open-end box, buried wall, sand, C-4, small cracks
4.0	1.0	0.5	NONE	--	--	10.4	3.3	Z=1.2, open-end box, buried wall, sand, C-4, major damage, near breach
4.0	1.0	0.5	NONE	--	--	0	0	Z=2.0, open-end box, buried wall, sand, C-4, small cracks
4.0	1.0	0.5	NONE	--	--	29.1	10	Z=1.5, open-end box, buried wall, sand, C-4, breach
4.0	1.0	0.5	NONE	--	--	1.6	0.5	Z=1.9, open-end box, buried wall, sand, C-4, small cracks
4.0	1.0	0.5	NONE	--	--	29.1	10	Z=1.4, open-end box, buried wall, sand, C-4, no comment
4.0	1.0	0.5	NONE	--	--	0	0	Z=1.5, open-end box, buried wall, sand, C-4, slight cracks
4.0	1.0	0.5	NONE	--	--	7.1	1.5	Z=1.0, open-end box, buried wall, sand, C-4, cracked concrete
4.0	1.0	0.5	NONE	--	--	26.6	6	Z=0.75, open-end box, buried wall, sand, C-4, severe concrete damage
4.0	1.0	0.5	NONE	--	--	c	c	Z=0.5, open-end box, buried wall, sand, C-4, breach
4.0	1.0	0.5	NONE	--	--	1.8	1.1	Z=2.8, open-end box, buried wall, sand, C-4, cracks
4.0	1.0	0.5	NONE	--	--	10.2	6.5	Z=2.3, open-end box, buried wall, sand, C-4, breach
4.0	1.0	0.5	NONE	--	--	c	c	Z=2.3, open-end box, buried wall, sand, C-4, breach
4.0	1.0	0.5	NONE	--	--	0	0	Z=1.86, closed-end box, buried wall, sand, C-4, slight cracks
4.0	1.0	0.5	NONE	--	--	2.2	1.4	Z=1.16, closed-end box, buried wall, sand, C-4, rear spalling
4.0	1.0	0.5	NONE	--	--	c	c	Z=0.70, closed-end box, buried wall, sand, C-4, breach
5.60	0.71	0.50	double	1.50	0.71	1.20	0.50	DOB = L/2, 3-H, Pso = 1812, no steel broken.
5.60	0.71	0.50	135-S-90	1.50	0.71	c	c	DOB = L/2, S, all steel broken @ supports, none @ midspan, Pso = 9000.
5.60	0.71	0.50	135-S-90	1.50	0.71	14.00	6.00	DOB = L/2, 3-H, Pso = 2176, 5% tension @ midspan rupture.
5.60	0.71	1.13	135-S-90	1.50	0.71	26.60	12.00	DOB = L/5, 3-H, Pso = 1900, 10% ten @ midspan, 40% ten & 20% comp @ supp
13.50	0.41	0.50	135-S-90	1.50	0.41	7.50	3.25	DOB = L/5, S, Pso = 11,500, no steel broke.
5.60	0.71	0.50	135-S-90	1.50	0.71	c	c	DOB = L/2, 3-HH, Pso = 8092, 60% ten @ midspan, 95% ten & 45% comp @ sup
5.60	0.71	0.50	135-S-90	1.50	0.71	c	c	DOB = L/5, 3-HH, Pso = 2364, 95% tension and compression rupture @ suppo
5.60	0.71	0.50	double	1.50	0.71	c	c	DOB = L/5, S, Pso = 4109, 27% ten & 14% comp @ supp rupt, remain. bars r
5.60	0.71	0.50	135-S-90	1.50	0.71	c	c	DOB = L/5, S, Pso = 5664, 9% ten rupt @ supp, remaining bars pulled out.
5.60	0.71	0.50	135-S-90	1.50	0.71	22.60	10.00	DOB = L/5, S, Pso = 3333, no steel rupture.
5.60	0.71	0.50	135-S-90	1.50	0.71	c	c	DOB = L/5, S, Pso = 4031, 73% ten & 45% comp rupt @ supp, remain. bars r
5.60	0.71	0.50	135-S-90	1.50	0.71	c	c	DOB = L/5, S, Pso = 6025, 68% tens & 55% comp rupture at support.
7.30	0.55	0.50	135-S-90	0.50	0.22	c	c	DOB = L/5, S, Pso = 7624, 29% tens & 11% comp rupture @ support.
7.30	0.55	0.50	135-S-90	0.50	0.22	c	c	DOB = L/5, S, Pso = 5682, 46% tens & 21% comp rupture at support.
7.30	0.55	0.50	135-S-90	0.50	0.22	10.40	4.13	DOB = L/5, S, Pso = 3448, no steel broke.
7.30	0.55	0.63	135-S-90	0.50	0.22	c	c	DOB = L/5, S, Pso = 8875, 7% tens rupture @ support, remaining bars pull
7.30	0.55	0.63	135-S-90	0.50	0.22	28.20	12.00	DOB = L/5, S, Pso = 5034, no steel broke.
7.30	0.55	0.63	135-S-90	0.50	0.22	8.90	3.50	DOB = L/5, S, Pso = 3377, no steel broke.
2.90	0.69	0.18	closed hoop	0.25	0.69	b	b	DOB = L/2, Pso = 3300, steel rupture undetermined.
2.90	0.69	0.18	closed hoop	0.25	0.69	3.60	0.75	DOB = L/2, Pso = 860, no steel broke.
2.50	1.50	0.25	double 135	0.18	0.60	0.90	0.25	DOB = 4L/11, 3-H, Pso = 127, no steel broke.
2.50	1.50	0.25	double 135	0.18	0.60	c	c	DOB = 0, Pso = 129, 100% tens & comp @ midspan md, 100% tens @ support r wall failed near mid-height.
2.50	1.50	0.25	double 135	0.18	0.60	0.20	0.06	DOB = 4L/11, Pso = 34, no steel broke.
2.50	1.50	0.25	double 135	0.18	0.60	1.70	0.50	DOB = 4L/11, Pso = 142, no steel broke.
2.50	1.50	0.25	double 135	0.18	0.60	3.10	0.88	DOB = 4L/11, Pso = 158, no steel broke.
2.50	1.50	0.25	double 135	0.18	0.60	2.40	0.69	DOB = 4L/11, Pso = 141, no steel broke.
2.50	1.50	0.25	double 135	0.18	0.60	2.30	0.66	DOB = 4L/11, Pso = 134, no steel broke.
2.50	1.50	0.25	double 135	0.18	0.60	1.70	0.50	DOB = 4L/11, Pso = 134, no steel broke.
2.25	1.00	0.20	--	--	--	1.40	0.41	DOB = 4L/11, Pso = 120, steel rupture @ support undetermined.
2.25	1.08	0.20	--	--	--	15.30	4.50	DOB = 4L/11, Pso = 184, 100% tens @ midspan rupture, undetermined @ supp
2.25	1.08	0.20	--	--	--	29.90	9.50	DOB = 4L/11, Pso = 128, 100% tens @ midspan rupture, undetermined @ supp
2.25	1.08	0.20	--	--	--	0.90	0.25	DOB = 4L/11, Pso = 162, steel rupture undetermined @ support.
6.0	0.67	0.18	135-S-90	0.31	0.67	16	11.4	Third layer of steel @ mid-depth; Ps=0.16 near midspan. Stirrups rupture or straightened @ support. Tens mebr. No principal steel rupture.
4.3	0.69	0.25	135-S-135	0.32	0.59	1.01	0.13	Z=4.0, max defl=0.38, buried wall, external shot, response mode undefine
4.3	0.69	0.25	135-S-135	0.32	0.59	5.66	1.5	Z=2.0, max defl=2.13, buried wall, external shot, hinged mode
4.3	0.69	0.25	135-S-135	0.32	0.59	c	c	Z=1.5, max defl=breach, buried wall, external shot, flexual mode
4.3	0.69	0.38	135-S-135	0.50	0.59	4.15	1.0	Z=2.0, max defl=1.56, buried wall, external shot, undefined mode
4.3	0.69	0.38	135-S-135	0.50	0.59	9.58	2.75	Z=1.5, max defl=3.63, buried wall, external shot, flexure-membrane mode
8.6	0.35	0.38	135-S-135	0.27	0.35	0.83	0.38	Z=2.0, max defl=0.63, buried wall, external shot, undefined mode
8.6	0.35	0.38	135-S-135	0.27	0.35	4.65	2.88	Z=1.0, max defl=3.5, buried wall, external shot, flexure-membrane mode
4.3	0.69	0.25	135-S-135	0.32	0.59	2.66	0.63	Z=2.0, max defl=1.0, buried wall, external shot, flexure mode
4.3	0.69	0.25	135-S-135	0.32	0.59	2.50	0.44	Z=2.0, mx defl=0.94, buried wall, external shot, flexure mode
4.3	0.69	0.25	135-S-135	0.32	0.59	6.84	2.13	Z=2.0, max defl=2.58, buried wall, external shot, flexure mode
4.3	0.69	0.25	135-S-135	0.32	0.59	4.49	1.19	Z=2.0, max defl=1.69, buried wall, external shot, flexure mode
10.30	0.58	0.75	NONE	--	--	14.00	17.00	Full scale, thin steel decking on bottom surface; HEST-160 psi; DOB = 4"
4.3	0.7	0.38	135-S-135	0.50	0.70	c	c	Z=2.0, wall buried in reconstituted clay, breach (hole) with 19" defl, many broken bars
8.6	0.35	0.38	135-S-135	0.28	0.28	2.1	1.19	Z=2.0, wall buried in reconstituted clay, light damage, cracking, max defl=1.56"
4.3	0.7	0.38	135-S-135	0.50	0.70	3.8	1.13	Z=2.0, wall buried in compacted sand, light damage, small cracks, max defl=1.44"
4.3	0.7	0.38	135-S-135	0.50	0.70	26.4	9.19	Z=2.0, wall buried in in-situ clay, most tension steel broken, max defl=10.69, tens. membrane, most cover hung on

CHAPTER 5: RESPONSE LIMITS

General

78. Portions of the data base from Tables 4.1 through 4.4 are categorized in Tables 5.1 through 5.5 to assist in the development of new design guidelines pertaining to response limits. The following discussions refer to the parameters that are emphasized by Tables 5.1 through 5.5.

Laterally Restrained Slabs

79. The roof, floor, and wall slabs of protective structures, particularly those in the data base, are generally laterally restrained. This is partly due to the extension of the principal reinforcement of a slab into the adjoining slab. Also, the adjacent slabs usually exhibit similar degrees of stiffness. Lateral restraint is necessary for the formation of tension membrane forces that enhance the large-deflection behavior of slabs. The laterally-restrained boxes tested at $z < 2.0\ ft/lb^{1/3}$ were all buried and had a tension steel reinforcement percentage (p) of 2.0 percent. For low values of L/d in the range of approximately 6 or 7 with $z = 1.0\ ft/lb^{1/3}$, damage was slight, but support rotations (Θ) were low (5 to 7 degrees) even when no shear reinforcement was used. Generally, wall slabs of boxes having L/d values of approximately 10 to 15 experienced large support rotations (15 to 29 degrees) and were damaged to near incipient collapse. However, a wall slab that had L/d = 7 and was tested at $z = 0.75\ ft/lb^{1/3}$ sustained a support rotation of 26 degrees without breaching, although there was no shear reinforcement. Breaching did not occur in this group of slabs until support rotations reached 15 degrees, and some slabs achieved support rotations significantly greater that 15 degrees without breaching occurring. In general, no shear

reinforcement was used in this group of slabs.

80. Many of the nonlaced slabs were tested in reaction devices of which the degree of lateral restraint cannot be determined with great confidence based on the information provided in the reports on the tests. Only two of the one-way slabs tested at $z < 2.0$ ft/lb$^{1/3}$ were definitely laterally restrained. Although one of these was lightly reinforced ($p = 0.15$) with no shear reinforcement and with L/d approximately equal to 9, it sustained only "slight" damage when tested at $z = 1.0$ ft/lb$^{1/3}$. Unfortunately, values for support rotation or midspan deflection are not available for these slabs. Damage was described as "heavy" when z was increased to 1.25 ft/lb$^{1/3}$, L/d was decreased to approximately 7, p was increased to 0.65, and looped reinforcement was used. Such variations in the data base are difficult to explain.

81. A considerable amount of information is available for the two way slabs that were laterally restrained with L/d greater than 20 and were tested at $z = 2.0$ ft/lb$^{1/3}$. The values of p for these slabs (0.31, 1.0, 1.5, and 2.5 percent) included low, middle, and high values, considering the range of p for the data base. For $p = 1.0$ or 1.5 percent, the slabs achieved support rotations of 10 to 12 degrees with no failure of the tension steel and "medium" damage. Even the slab having the low value of $p = 0.31$ percent with no stirrups sustained a support rotation of 10.4 degrees with medium damage and no rupture of reinforcement. The support rotation was limited to 5 degrees due to the high percentage of principal reinforcement when p equaled 2.5 percent. The slabs that sustained large deflections did not experience breaching, although z was as low as 0.65 ft/lb$^{1/3}$. When the single-leg

stirrups (180-degree bends on each end) were used, they were spaced at less than one-half the thickness of the slab.

82. A review of data for the laterally-restrained lace slabs tested at $z < 2.0$ ft/lb$^{1/3}$ provides some insight into the difference in the behavior of laced and nonlaced slabs. The fact that both a laced slab and a slab with no shear reinforcement incurred heavy damage when tested at $z = 1.5$ ft/lb$^{1/3}$ and 1.25 ft/lb$^{1/3}$ respectively, somewhat questions the significance of lacing. When laced slabs with $p = 2.7$ percent were subjected to low z values of 0.3 and 0.5 ft/lb$^{1/3}$, they experienced heavy damage and partial destruction, respectively. It is interesting to note that a laterally-unrestrained slab with no shear reinforcement and $p = 2.7$ incurred only medium damage at $z = 0.5$ ft/lb$^{1/3}$. This indicates that the effects of the large p of 2.7 percent overshadowed the effects of shear reinforcement on the response of these slabs.

83. The data base also includes a group of laterally-restrained slabs (components of box structures) tested at $z = 2.0$ ft/lb$^{1/3}$. The L/d values for these slabs ranged from approximately 6 to 20 and p was relatively large, 2.0 percent (the upper limit of TM 5-855-1). Support rotations were generally small and the damage was slight (mainly hairline cracks). Support rotations were as high as 26 degrees for a wall slab of a box buried in clay. Typically, the boxes in the data base were buried in sand, which is generally known to result in less structural response than when clay backfill is used. A slab with a L/d value approximately 6 incurred only slight damage with a support rotation of 2 degrees when z equaled 2.0 ft/lb$^{1/3}$. This slab contained single-leg stirrups, with 135-degree bends on each end, spaced at

less than one-half the slab thickness. The slab that was tested in clay contained similar stirrups spaced at greater than one-half the slab thickness. As z was increased to 2.8, 4.0, and 5.0 ft/lb$^{1/3}$ for some walls, support rotations remained very small (1.5, 1.0, and 2.0 degrees).

84. Although many of the HEST tests are often considered to be "highly-impulsive", it is likely that they may more accurately represent tests that have a charge placed at $z \geq 2.0$ ft/lb$^{1/3}$. The parameter p varied from 0.5 to 1.2 percent and the boxes usually contained single-leg stirrups with a 90-degree bend on one end and a 135-degree bend on the other end. The stirrups were spaced at less than one-half the slab thickness and the L/d values ranged from approximately 7 to 17. Generally, very little steel was ruptured in these tests. The only case in which more than 50 percent of the tension reinforcement was ruptured was for a slab with no shear reinforcement and p = 1.2 percent. Also, the principal reinforcement was spaced at greater than the slab thickness and the slab experienced support rotations of 15 degrees. When the principal reinforcement in a similar slab (p = 1.1 percent) was spaced at less than the slab thickness, no steel was ruptured. This slab sustained support rotations of 14 degrees. In addition, a slab with single-leg stirrups (90- and 135- degree bends), p of only 0.51 percent (spacing less than the slab thickness), and L/d of approximately 15 achieved support rotations of 16 degrees with no rupture of steel. This group of data indicates that slabs with single-leg stirrups (90- and 135- degree bends) and L/d values from 7 to 17 are capable of sustaining support rotations up to 30 degrees with significant damage and can achieve support rotations of approximately 25 degrees with little to no rupture of steel. Actually, this was the case for some slabs that contained no shear reinforcement.

Laterally-Unrestrained Slabs

85. Data for laterally-unrestrained, nonlaced slabs tested at $z < 2.0$ ft/lb$^{1/3}$ are very limited. One of these slabs contained looped shear reinforcement, had an L/d value of approximately 7, and was tested at $z = 1.0$ ft/lb$^{1/3}$. The damage was described as partial destruction. The rest of the slabs in the data base for this category contained no shear reinforcement. The damage levels ranged from slight damage to total destruction for slabs that had an L/d of approximately 10, a p of 0.15 percent, and were tested at z values from 1.7 to 1.0 ft/lb$^{1/3}$. Medium damage occurred when z equaled 1.1 ft/lb$^{1/3}$. When slabs having L/d of approximately 7 were tested at $z = 0.5$ ft/lb$^{1/3}$, one with $p = 0.65$ percent incurred total destruction, and one with $p = 2.7$ percent incurred medium damage. Likewise, an unrestrained laced slab with $p = 2.7$ percent incurred heavy damage when tested at $z = 0.5$ ft/lb$^{1/3}$. Damage was also heavy for two unrestrained laced slabs with $L/d = 7$ and $p = 0.65$ percent when tested at $z = 1.0$ ft/lb$^{1/3}$. It is obvious that unrestrained slabs with low percentages of tension steel are susceptible to major damage when $z < 2.0$ ft/lb$^{1/3}$.

86. Data for laterally-unrestrained, nonlaced slabs tested at $z \geq 2.0$ ft/lb$^{1/3}$ are also very limited. Four of these slabs had an L/d of approximately 10 and a very low p of 0.15 percent. The damage levels ranged from total destruction when z equaled 2.0 ft/lb$^{1/3}$ to slight damage when z equaled 2.6 ft/lb$^{1/3}$. Slight damage also occurred when L/d was approximately 14, p equaled 0.4 percent, and z equaled the relatively large value of 3.5 ft/lb$^{1/3}$. All of these one-way slabs contained no shear reinforcement.

RESPONSE LIMITS

87. Much of the data discussed in this report were taken from tests on walls or roofs of buried box structures. Other above-ground tests were typically conducted using bare (uncased) explosives, which did not produce a fragment loading and consequent degradation of the slabs. This study supports the development of new shear reinforcement design criteria and associated response limits for protective structures designed to resist the effects of conventional weapons. Based on this data review, recommended response limits are given in Table 5.6.

88. As discussed throughout this report, laterally unrestrained and laterally restrained slabs behave differently because tension membrane forces can develop in a one-way slab only if the slab is laterally restrained at the supports. However, lateral restraint is inherent to two-way slabs. Table 5.6 presents allowable support rotations for laterally restrained slabs based on acceptable damage levels that must be chosen by the designer, depending on the purpose of the structure. Moderate damage means that significant concrete scabbing and reinforcement rupture has not occurred and the dust and debris environment on the protected side of the slab is moderate; however, large slab motions will occur. Such a damage level may be acceptable for the protection of personnel and sensitive equipment. Heavy damage means that the slab is at incipient failure, and significant reinforcement rupture may have occurred over much of the slab. In this case, the slab may resemble a reinforcing grid suspending concrete rubble.

89. Based on the limits of the data base, the response limits given in Table 5.6 should only be used if: (1) the scaled range exceeds 0.5 $ft/lb^{1/3}$, (2)

the clear span to effective depth ratio (L/d) exceeds 5, (3) the principal reinforcement spacing is minimized (never exceeding the effective depth of the slab), and (4) adequate stirrups are provided. Stirrup reinforcement is required to provide adequate concrete confinement and principal steel support in the large-deflection region. Stirrups should be required along each principal reinforcing bar at a maximum spacing of d/2 when $z < 2$ ft/lb$^{1/3}$, and at a maximum spacing of d at larger scaled ranges. When stirrups are required to resist shear, stirrup spacing should not exceed d/2. In accordance with Reference 5, all stirrup reinforcement should provide a minimum of 50 psi shear stress capacity. Single-leg stirrups having a 135-degree bend on one end and at least a 90-degree bend on the other end are recommended for economy.

90. It is observed from the data base that flexible slabs that are laterally restrained are much less likely to fail in direct shear because early in the response, lateral compression membrane forces will act to increase the shear capacity, and later in the response shear forces tend to be resolved into the principal reinforcement during tension membrane action. Tests indicate that direct shear failure can occur in slabs subjected to impulsive loads. It is generally known that shear-type failure is more likely to occur in reinforced concrete members with small L/d values than it is in those with large L/d values. Since the data base indicates that laterally restrained slabs with $L/d \geq 8$ are unlikely to experience direct shear failures, consideration for the design of details to resist direct shear are only recommended for laterally restrained slabs having $L/d < 8$ and for all laterally unrestrained slabs. This is considered to be conservative, but the degree of conservatism is unknown due to gaps in the data base.

TABLE 5.1.
LATERALLY-RESTRAINED BOXES

S = principal steel spacing
U = not reported (unknown)
S_s = shear reinforcement spacing
t = slab thickness

z	L/t	Θ	Shear Rein.	$s \le t$	$s_s \le t/2$	Damage
1.5	8	29	None	Y	---	Local Breach
1.4	8	28	None	Y	---	U
0.75	6	26	None	Y	---	U
1.9	12	15	None	Y	---	Local Breach
1.2	9	10	None	Y	---	Major Damage
1.5	10	10	135-s-135	Y	N	U
1.2	12	7	None	Y	---	Major Damage
1.0	6	7	None	Y	---	Slight
1.16	18	2	None	Y	---	Slight
1.8	12	2	None	Y	---	Slight
1.8	9	1	None	Y	---	Slight
1.86	18	0	None	Y	---	Slight
1.5	6	0	None	Y	---	Slight
1.0	5	5	135-s-135	Y	Y	U
1.9	9	2	None	Y	---	Slight
			$z \ge 2.0$			
2.0	10	26	135-s-135	Y	N	U
2.3	18	10	None	Y	---	Local Breach
2.0	10	7	135-s-135	Y	N	U
2.0	10	6	135-s-135	Y	N	U
2.0	10	4.5	135-s-135	Y	N	U
2.0	10	4	135-s-135	Y	N	U
2.0	10	3.5	135-s-135	Y	N	Slight

TABLE 5.1. LATERALLY-RESTRAINED BOXES (cont'd)

$z < 2.0$

z	L/t	Θ	Shear Rein.	$s \le t$	$s_s \le t/2$	Damage
2.0	10	2.5	135-s-135	Y	N	U
2.0	12	2.5	None	Y	---	Slight
2.0	10	2	135-s-135	Y	N	U
2.0	5	2	135-s-135	Y	Y	Slight
2.8	18	1.5	None	Y	---	Slight
4.0	10	1	135-s-135	Y	---	Slight
2.3	12	1	None	Y	---	Slight
2.0	5	1	135-s-135	Y	Y	U
2.4	12	0.5	None	Y	---	Slight
5.0	7	0.2	None	Y	---	Slight
2.0	9	0	None	Y	---	Slight

HEST LOADING

L/t	Θ	Shear Rein.	$s \le t$	$p_{ten} \backslash p_s$	$s_s \le t/2$	Damage
15	30	None	N	1.2	---	near incipient collapse
6	28	135-s-90	Y	1.2\0.5	Y	steel not ruptured
8	26	135-s-90	Y	1.0\1.5	N	< 50% steel ruptured
8	22	135-s-90	Y	1.0\1.5	N	steel not ruptured
14	16	135-s-90	Y	0.51\0.31	N	steel not ruptured

HEST LOADING (cont'd)

L/t	Θ	Shear Rein.	$s \le t$	$p_{ten}\backslash p_s$	$s_s \le t/2$	Damage
15	15	None	N	1.2	---	> 50% steel ruptured
13	14	None	Y	1.1	---	steel not ruptured
8	14	135-s-90	Y	1.0\1.5	N	< 10% steel ruptured
6	11	135-s-90	Y	0.75\0.5	Y	steel not ruptured
6	9	135-s-90	Y	1.2\0.5	Y	steel not ruptured
8	8	135-s-90	Y	1.5\1.5	Y	steel not ruptured
8	4	closed-hoop	Y	0.5\0.25	---	steel not ruptured
13	3.1	double-leg	N	0.69\0.18	N	steel not ruptured
13	2.5	double-leg	N	0.69\0.18	N	steel not ruptured
13	2	double-leg	N	0.69\0.18	N	steel not ruptured
13	2	double-leg	N	0.69\0.18	N	steel not ruptured
8.5	1.5	double-leg	Y	1.0\1.5	N	steel not ruptured
15	1.5	None	N	1.2	---	< 10% steel ruptured
13	1	double-leg	N	0.69\0.18	N	steel not ruptured
15	1	None	N	1.2	---	< 10% steel ruptured
13	0.5	double-leg	N	0.69\0.18	N	steel not ruptured

TABLE 5.2. NONLACED SLABS

SD = Slight damage
MD = Medium damage
HD = Heavy damage
PD = Partial destruction
TD = Total damage

$z < 2.0$

z	L/t	Shear Rein.	$s \le t$	$p_{tension}$%	Laterally Restrained	Damage
1.7	8	None	Y	0.15	N	SD
1.7	8	None	Y	0.15	N	SD
1.65	8	None	Y	0.15	N	PD
1.6	6	None	N	0.65	U	PD
1.5	8	None	Y	0.15	U	TD
1.5	14	None	Y	0.40	U	SD
1.5	14	None	Y	0.40	U	HD
1.25	6	None	N	0.65	U	TD
1.25	6	None	N	0.44	U	HD
1.25	6	None	N	0.65	U	HD
1.25	6	None	N	0.65	U	PD
1.25	6	Looped	N	0.65	Y	HD
1.1	8	None	Y	0.15	N	MD
1.05	8	None	Y	0.15	N	PD
1.02	7	None	Y	0.15	U	TD
1.0	8	None	Y	0.15	N	TD
1.0	8	None	Y	0.15	N	TD
1.0	7	None	Y	0.15	Y	SD
1.0	6	None	N	0.65	U	TD
1.0	6	Looped	N	0.65	N	PD
0.8	6	None	N	0.65	U	TD
0.5	14	None	Y	0.40	U	TD
0.5	8	None	Y	0.15	N	TD
0.5	6	None	N	0.65	U	HD

TABLE 5.2. NONLACED SLABS (cont'd)

$z < 2.0$

z	L/t	Shear Rein.	$s \le t$	$p_{tension}$%	Laterally Restrained	Damage
0.5	6	None	N	0.44	U	TD
0.5	6	None	N	0.65	U	HD
0.5	6	None	N	0.65	U	TD
0.5	6	None	N	0.65	N	TD
0.5	4	None	N	2.70	N	MD
0.5	2	None	N	0.15	U	TD
1.1	20	None	Y	0.31	Y	$\Theta = 10.4°$; no steel failed shear crack @ one support (MD)
0.68	20	180-s-180	Y	1.0	Y	$\Theta = 12.2°$; no steel failed (MD)
0.68	20	180-s-180	Y	1.0	Y	$\Theta = 10.1°$; no steel failed (MD)
0.65	20	180-s-180	Y	1.5	Y	$\Theta = 10.5°$; no steel failed (MD)
0.65	20	180-s-180	Y	2.5	Y	$\Theta = 4.8°$; no steel failed (SD-MD)

$z \ge 2.0$

z	L/t	Shear Rein.	$s \le t$	$p_{tension}$%	Laterally Restrained	Damage
2.0	8	None	Y	0.15	U	TD
2.6	8	None	Y	0.15	N	SD
2.6	8	None	Y	0.15	N	PD
2.62	8	None	Y	0.15	N	SD
3.5	14	None	Y	0.40	U	SD

TABLE 5.3. LACED SLABS

z < 2.0

z	L/t	$p_{tension}$%	p_{shear}%	Laterally Restrained	Damage
1.5	6	0.65	0.15	Y	HD
1.25	6	0.65	0.40	U	MD
1.0	6	0.65	0.15	N	HD
1.0	6	0.65	0.40	N	HD
1.0	6	0.65	0.15	Y	HD
1.0	6	0.65	0.15	Y	PD
1.0	6	2.70	1.20	Y	HD
0.9	6	2.70	1.20	Y	HD
0.8	6	0.65	0.15	N	PD
0.8	6	0.65	0.40	N	MD
0.5	6	0.65	0.40	U	HD
0.5	6	0.65	0.15	U	PD
0.5	6	0.65	0.40	U	PD
0.5	6	2.70	1.20	N	HD
0.5	4	2.70	1.20	Y	HD
0.5	2	0.69	0.53	U	MD
0.4	6	0.65	0.40	U	HD
0.4	1.8	0.65	0.53	U	HD
0.35	2	2.70	1.20	Y	HD
0.3	2	2.70	1.20	Y	HD
0.3	2	2.70	1.20	Y	PD

TABLE 5.4. NONLACED SLABS
STATICALLY-LOADED

Θ	L/t	Shear Rein.	s ≤ t	s_s ≤ t/2	p_{ten}/p_s	Damage
11.2	15	135-s-90	N	N	1.14/0.18	> 50% tension steel ruptured
12.6	10	135-s-90	N	N	0.74/0.18	< 50% tension steel ruptured
13	10	135-s-135	N	N	0.74/0.09	> 50% tension steel ruptured
14	10	double-leg	N	N	0.74/0.19	> 50% tension steel ruptured
14	10	135-s-135	N	N	0.74/0.18	> 50% tension steel ruptured
14	10	135-s-90	N	N	0.74/0.18	> 50% tension steel ruptured
14	10	None	N	N	1.58	No steel ruptured
14.5	10	135-s-135	N	N	0.74/0.18	> 50% tension steel ruptured
14.5	15	135-s-90	N	N	1.47/0.24	No steel ruptured
15	15	135-s-90	N	N	1.47/0.24	No steel ruptured
15.5	10	135-s-135	N	N	0.74/0.18	> 50% tension steel ruptured
16	15	135-s-90	N	N	0.58/0.18	> 50% tension steel ruptured
16.5	10	135-s-90	N	N	1.06/0.27	< 50% tension steel ruptured
16.5	10	None	N	N	0.74	> 50% tension steel ruptured
16.5	10	135-s-135	Y	N	0.75/0.19	> 50% tension steel ruptured
16.7	15	135-s-90	N	N	1.14/0.18	No steel ruptured
17	10	135-s-90	N	N	0.52/0.22	> 50% tension steel ruptured
17	15	135-s-90	N	N	0.58/0.18	> 50% tension steel ruptured
8	10	135-s-90	N	N	0.74/0.18	< 50% tension steel ruptured
18	15	135-s-90	N	N	1.14/0.18	No steel ruptured
18	10	135-s-135	Y	Y	0.75/0.38	> 50% tension steel ruptured
18	10	None	N	N	0.74	> 50% tension steel ruptured
18.8	10	135-s-90	N	N	0.74/0.18	> 50% tension steel ruptured
19.5	10	135-s-90	N	Y	1.13/0.22	> 50% tension steel ruptured
19.5	10	135-s-90	N	N	0.52/0.22	> 50% tension steel ruptured
19.7	10	None	N	N	0.79	> 50% tension steel ruptured
19.7	10	None	N	N	1.13	< 50% tension steel ruptured

TABLE 5.4. NONLACED SLABS
STATICALLY-LOADED (cont'd)

Θ	L/t	Shear Rein.	$s \le t$	$s_s \le t/2$	p_{ten}/p_s	Damage
20	10	135-s-90	N	N	0.74/0.18	> 50% tension steel ruptured
20.5	10	135-s-135	N	Y	0.74/0.36	> 50% tension steel ruptured
20.5	10	None	N	N	1.14	< 50% tension steel ruptured
21	10	None	N	N	1.14	> 50% tension steel ruptured
22.5	10	None	N	N	1.13	< 50% tension steel ruptured
22.5	8.4	135-s-90	Y	N	1.02/1.53	> 50% tension steel ruptured
23.5	10	135-s-90	N	Y	1.13/0.22	> 50% tension steel ruptured
23.5	10	None	N	N	1.14	> 50% tension steel ruptured
23.5	10	135-s-135	N	N	1.13/0.06	> 50% tension steel ruptured
24	10	None	N	N	0.79	> 50% tension steel ruptured
24.5	10	135-s-90	N	Y	1.13/0.22	> 50% tension steel ruptured

TABLE 5.5. LACED SLABS
STATICALLY LOADED

Θ	L/t	p_{ten}/p_s	$s \le t$	$s_s \le t/2$	Laterally Restrained	Damage
8.5	24	0.82/0.19	N	Y	Y	steel condition not reported
9.2	24	2.11/1.37	Y	Y	Y	no steel ruptured
11	24	0.89/0.42	N	Y	Y	steel condition not reported
12.5	24	0.82/0.19	N	Y	Y	steel condition not reported
13.2	24	0.82/0.19	N	Y	Y	steel condition not reported

Table 5.6. Recommended Response Limits for Reinforced Concrete Slabs

Lateral Restraint Condition	Damage Level	Response Limit (Degrees)
Unrestrained	———	6
Restrained	Moderate	12
Restrained	Heavy	20

CHAPTER 6: TRUSS-MODEL ANALOGY

91. The state-of-the-art in truss-model analogy is being identified and studied for use as a tool in evaluating the effects of stirrups and lacing bars on the load-response behavior of reinforced concrete slabs. Hsu (Reference 31) gives a brief history of the truss model for shear. Ritter (Reference 32) and Morsch (Reference 33) developed the concept of simulating the post-cracking action of a reinforced concrete member by a truss model. Diagonal cracks will form in a reinforced concrete beam subjected to shear, and the concrete may be thought of as a series of separate concrete struts. The top and bottom longitudinal bars serve as the top and bottom chords of the truss. The transverse steel bars (such as stirrups) and the concrete struts serve as web members of the plane truss. The inclination of the concrete struts was assumed to be 45 degrees. The stresses in the transverse steel, in the longitudinal steel, and in the concrete struts can be obtained from equilibrium.

92. The truss-model analogy has been extended to include torsion, as well as shear and bending. Lampert and Thurlimann (Reference 34) assumed that the angle of inclination of the concrete struts may deviate from 45 degrees. They used the theory of plasticity and called their theory the variable-angle truss model.

93. Elfgren (Reference 35) further applied the variable-angle truss model to members subjected to torsion, bending, and shear. He compared the variable-angle truss model to Wagner's tensile field theory (Reference 36) for a metal girder. Since the concrete web in a reinforced concrete member is assumed to take only compressive stress after cracking, Elfgren called his theory the

"compressive stress field theory." His theory to determine the angle of the compressive stress field was based on the plasticity theory. However, Wagner's angle for a tensile stress field was derived from strain compatibility. Elfgren recognized that the angle of the compression field is different from the actual angle of the cracks.

94. Collins (References 377and 38) developed the variable-angle truss model using strain compatibility instead of plasticity theory. He derived a compatibility equation (identical to that of Wagner) to determine the angle of the compression stress field. The compatibility equation enables the strain to be predicted by Mohr's circle. Collins called his theory the "diagonal compression field theory."

95. The compressive stress-strain curve of the concrete struts must be assumed in addition to the compatibility and equilibrium equations in the variable-angle truss model. Hsu and Mo (Reference 38) found that the use of the conventional stress-strain curve obtained from the standard concrete compression cylinder leads to unconservative strength predictions. They proposed a "softened" stress-strain curve, which resulted from diagonal shear cracking, to correctly predict the torsional strength as well as the deformations and strains throughout the loading history.

96. In summary, the compression field theory is based on the variable-angle truss model, assuming that the angle of inclination of the cracks is identical to the inclination of the compression field. Lampert and Thurlimann's theory and Elfgren's theory are based on the theory of plasticity (plasticity compression field theory). Collins' theory and Hsu and Mo's theory can be called "compatibility compression field theory" since they use the strain compatibility of the truss model.

97. The application of these theories (particularly those of Collins and of Hsu and Mo) to slabs containing stirrups and slabs containing lacing bars will provide an analytical comparison of the two types of reinforcement. The use of some of the data previously presented in this report will aid in the evaluation of the usefulness of the theories in studying the effects of the shear reinforcement details. Depending on available funding, additional data will be generated by physical model testing in order to fill in the gaps of the current data base and to allow a complete evaluation of the theories for potential use in a design procedure.

CHAPTER 7: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Summary

98. Most of the effort thus for in this study was directed toward the collection of pertinent test data and the extraction of design and test parameters and test results. The state-of-the-art in the use of shear reinforcement in the design of structures to resist the effects of conventional weapons or the effects of accidental explosions in explosive manufacturing and storage facilities were discussed. The tests series were described, and the slab parameters and test results tabulated. Analytical theories to be further studied were identified. Suggested preliminary guidelines for shear reinforcement requirements based on response limits were presented.

Conclusions

99. The use of some type of shear reinforcement is uniformly required by current manuals for blast design. The design criteria of the current blast-resistant design manuals, particularly the widely used Draft TM 5-1300, appear to be overly conservative. The design criteria are based on an incomplete test series (practically no stirrup slab tests). Also, recent tests indicate that slabs with stirrups can sustain large support rotations, and that some slab parameters other than the standoff distance also contribute significantly to ductile behavior. The L/t ratio, principal steel spacing and percentage, and support conditions are examples of significant parameters.

100. The suggested preliminary guidelines for shear reinforcement requirements based on response limits constitute the first step toward the development of new design criteria. Additional data is needed for the development of a more accurate and less conservative design criteria. The use of analytical theories based on truss-model analogy has potential for the development of a design procedure that shculd be validated by test data.

Recommendations

86. An experimental program is needed to allow an update of current design criteria. Further study of the existing test data will help to optimize the design of a test program. Also, further study of the mechanics associated with lacing and stirrups within a slab (using truss-model analogy) will aid in the design of the experiments and the development of design criteria. A test program that includes both static and dynamic tests that will significantly benefit this study has been proposed to several interested agencies. Some data gaps need to be filled and perhaps proof tests need to be conducted before guidelines are developed that will result in more economical facilities used for explosives handling and storage.

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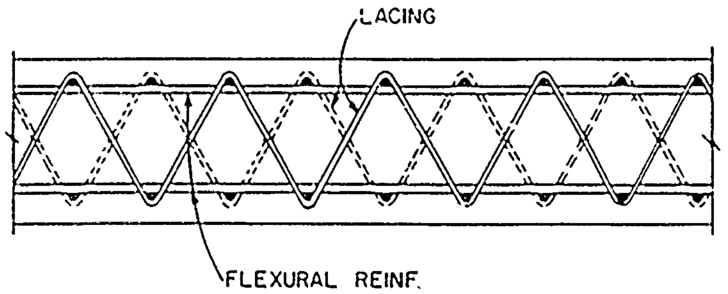
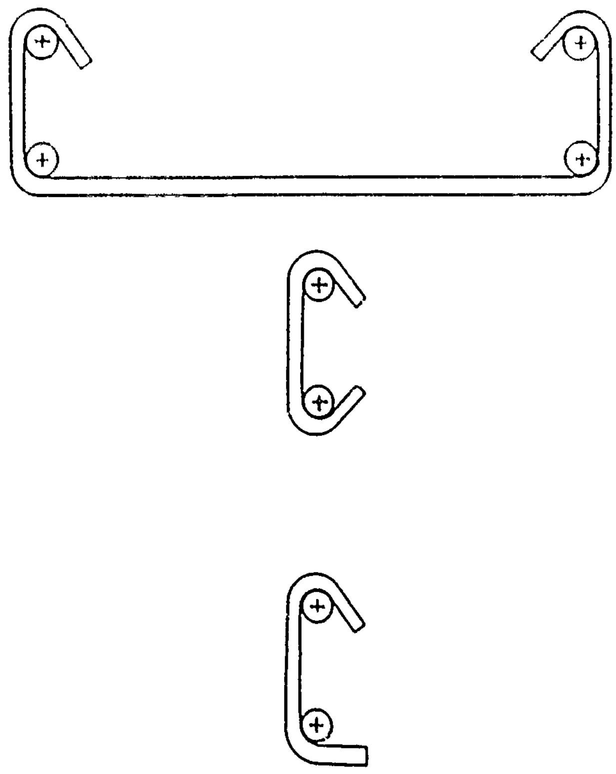


a. Lacing reinforcement



b. Stirrup configurations

Figure 1. Shear reinforcement

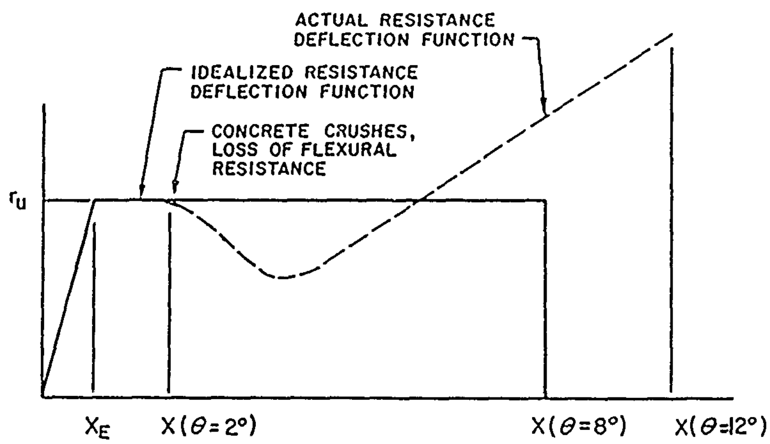


Figure 2. Idealized resistance deflection curve for large deflections

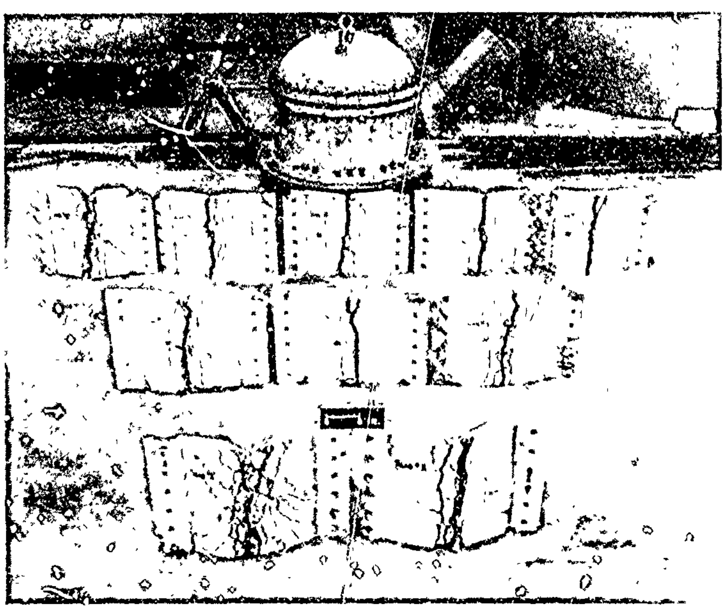


Figure 3. Posttest view of slabs with stirrups

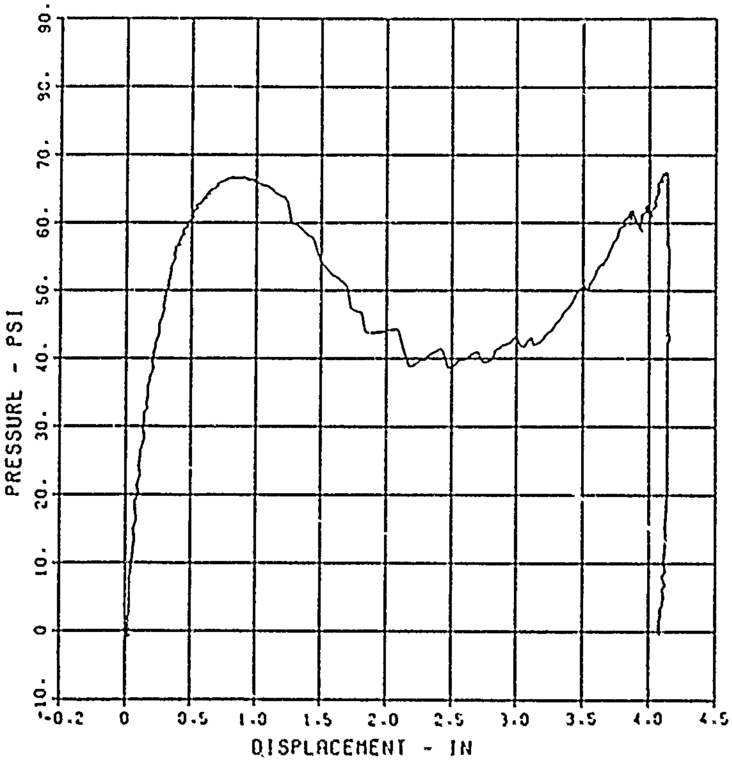


Figure 4. Load deflection curve for close stirrup spacing

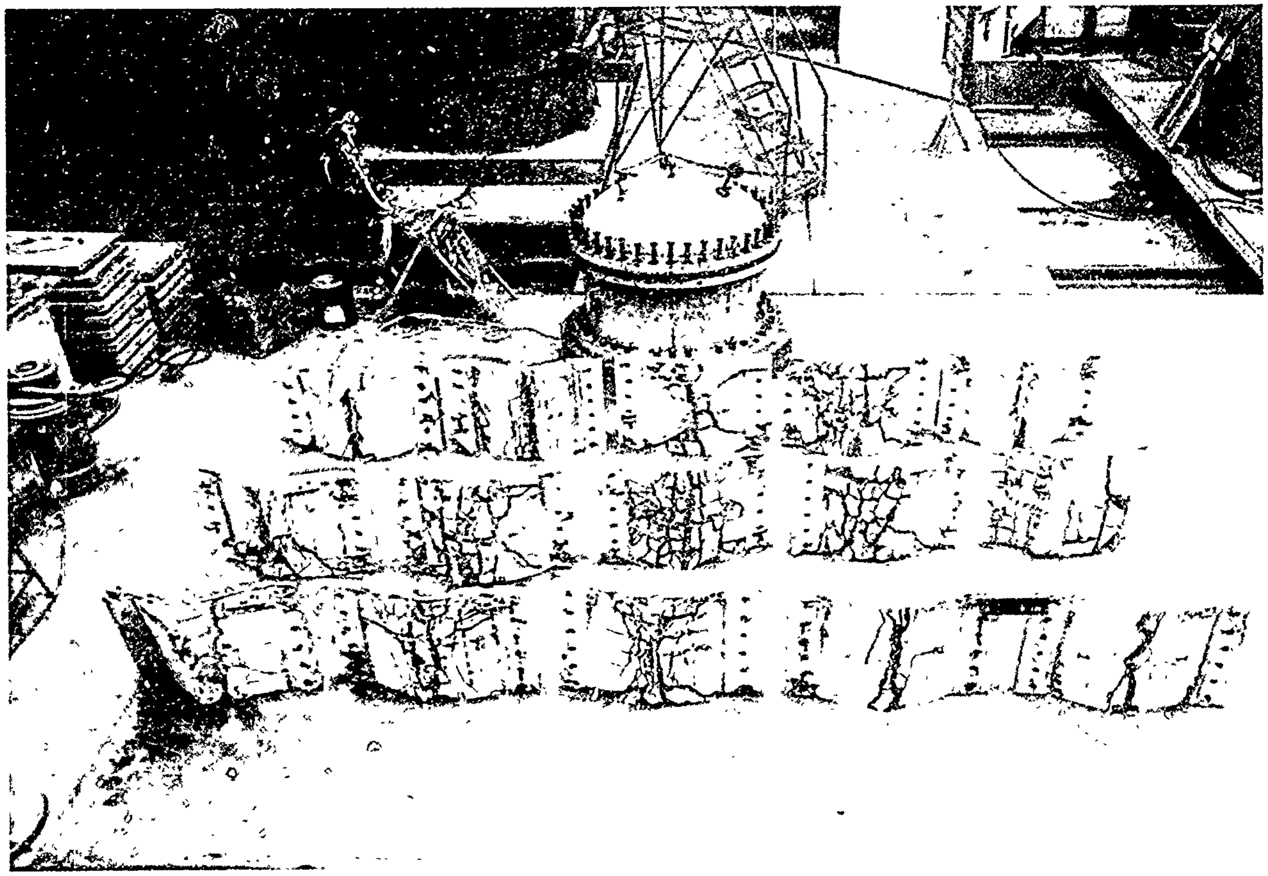


Figure 5. Posttest view of W-84 Series

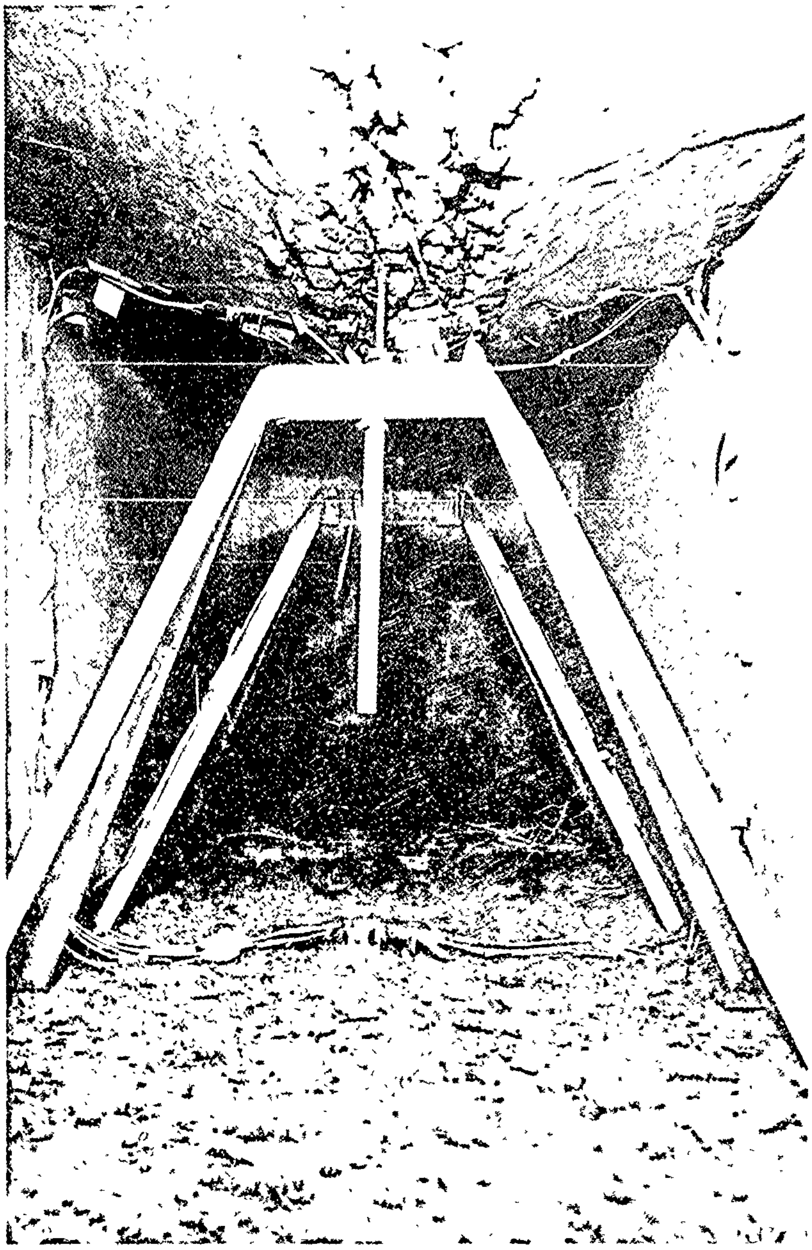
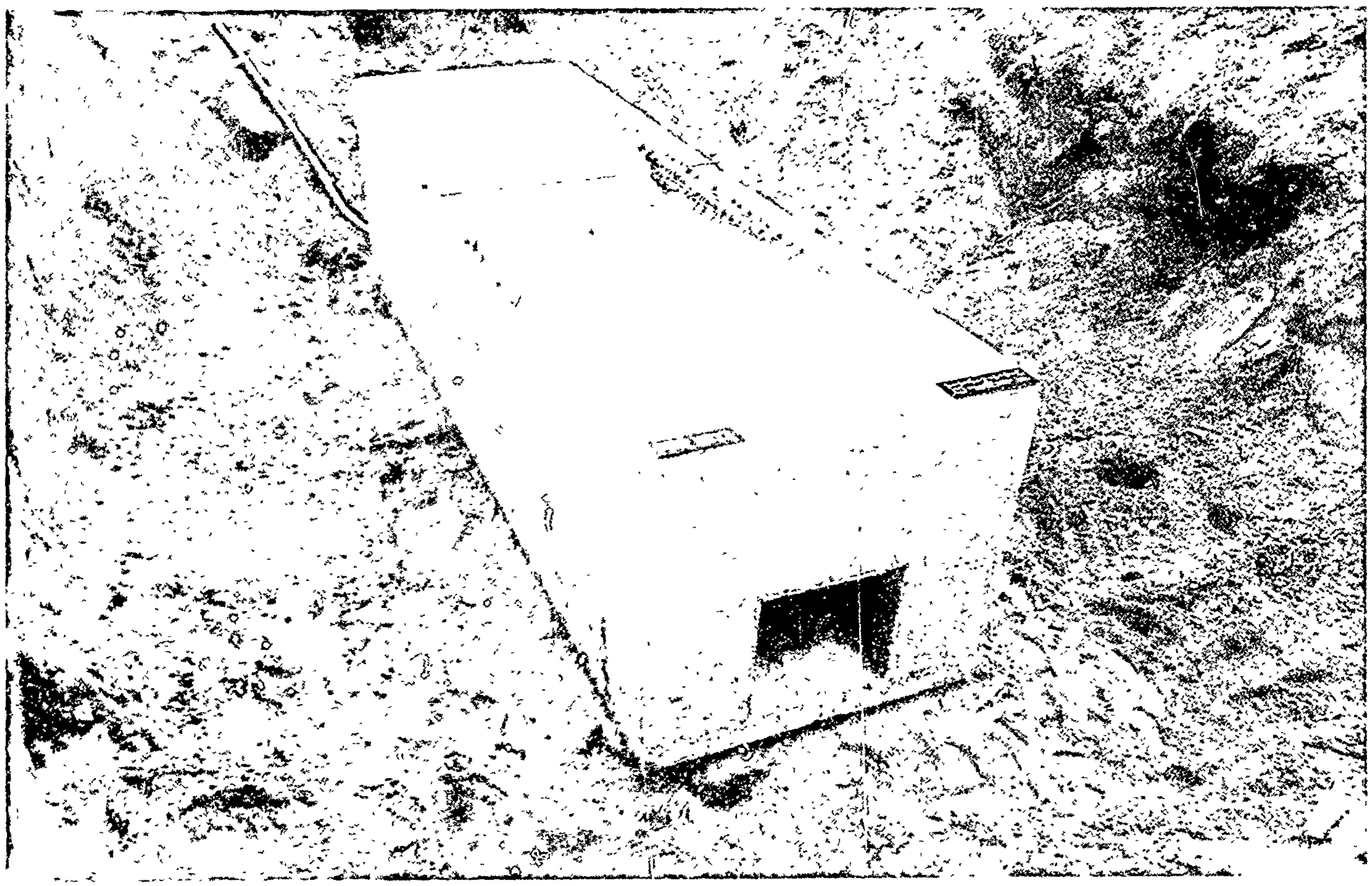


Figure 6. Damage to structure tested to clay backfill



ıre 7. Damage to structure tested in sand backfill

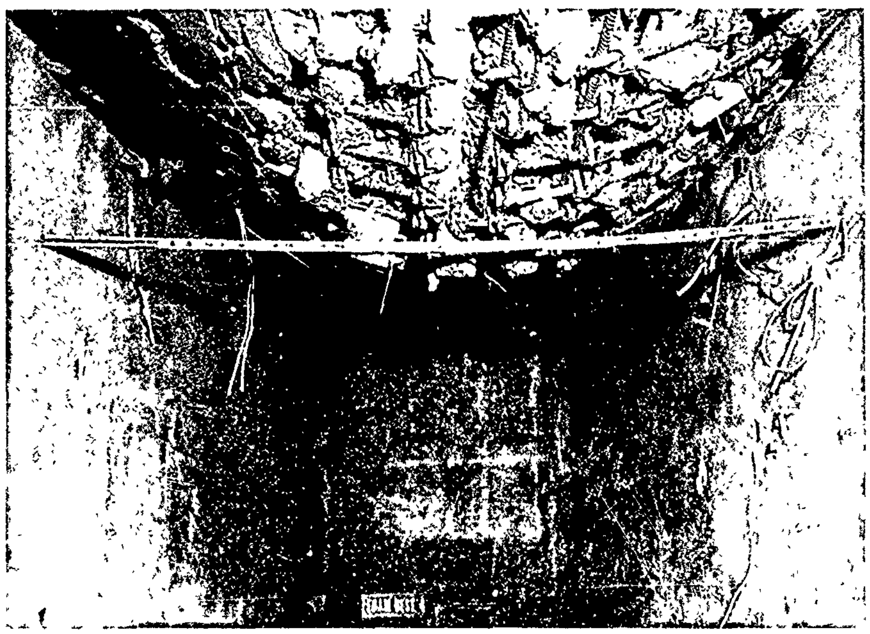


Figure 8. Interior view of structure tested in sand backfill

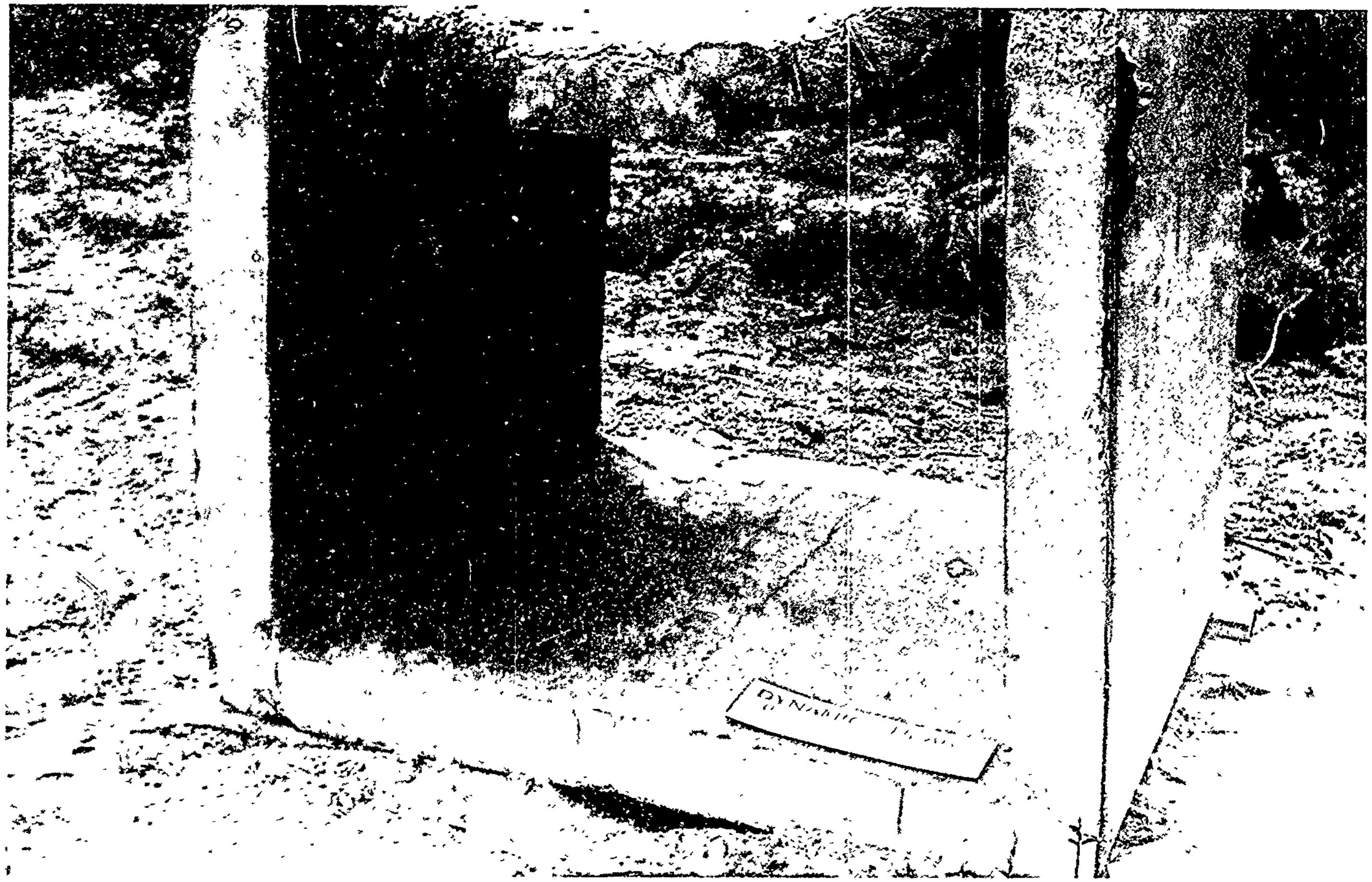


Figure 9. Shallow-buried box with 10-inch roof deflection

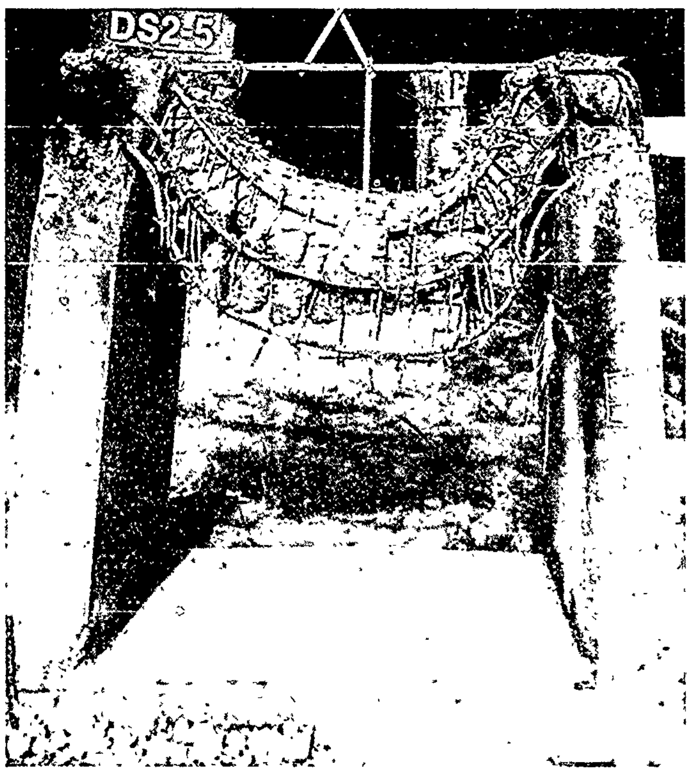


Figure 10. Shallow-buried box with 12-inch roof deflection

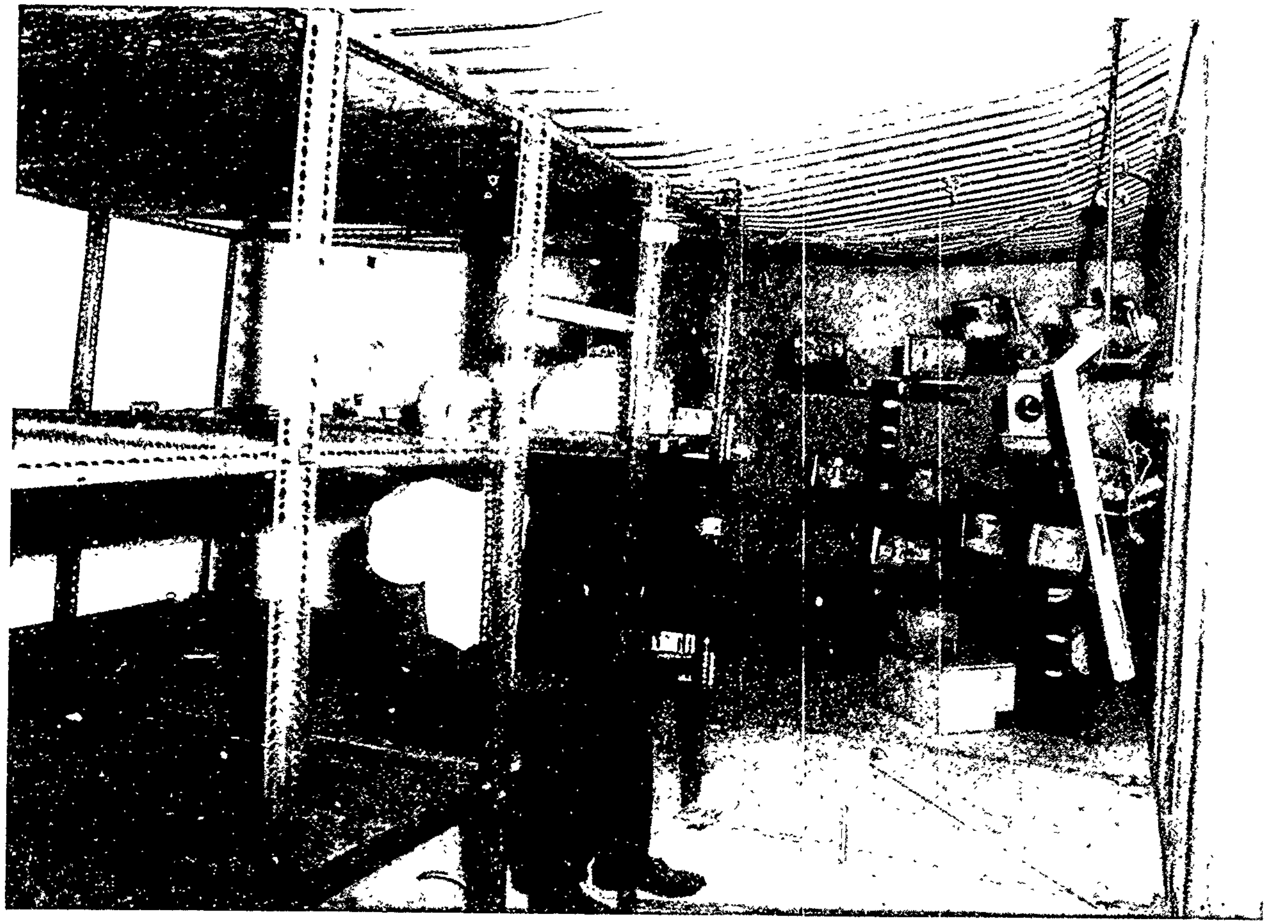


Figure 11. Interior view of KW-87

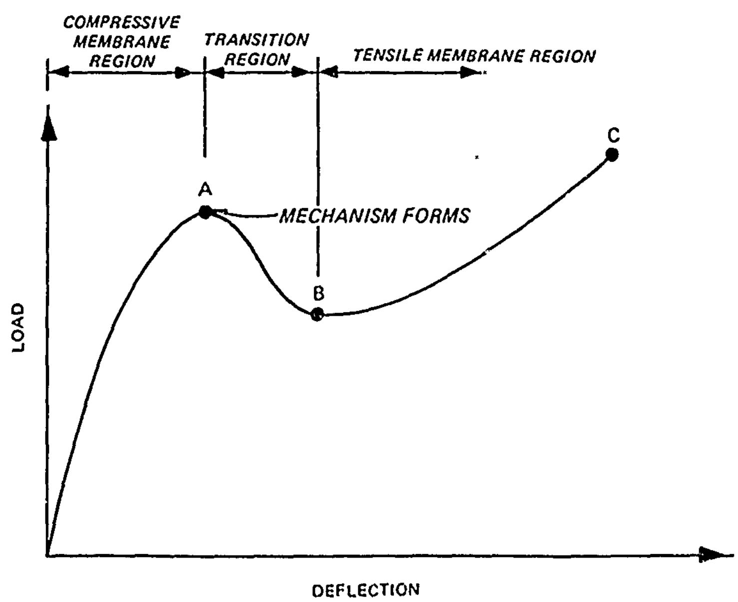


Figure 12. Load-deflection relationship for restrained slabs

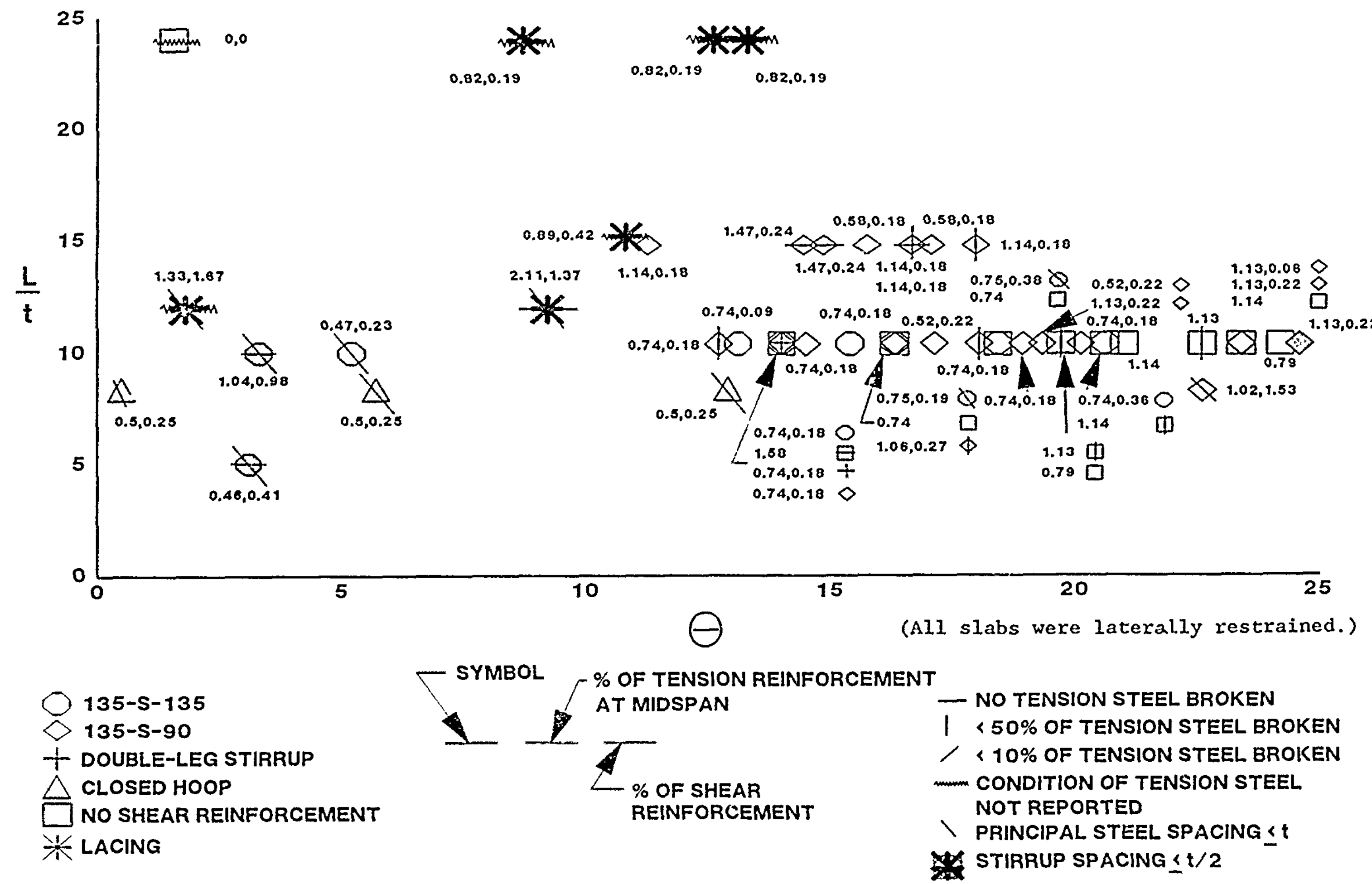


Figure 13. Statically-Loaded Slabs

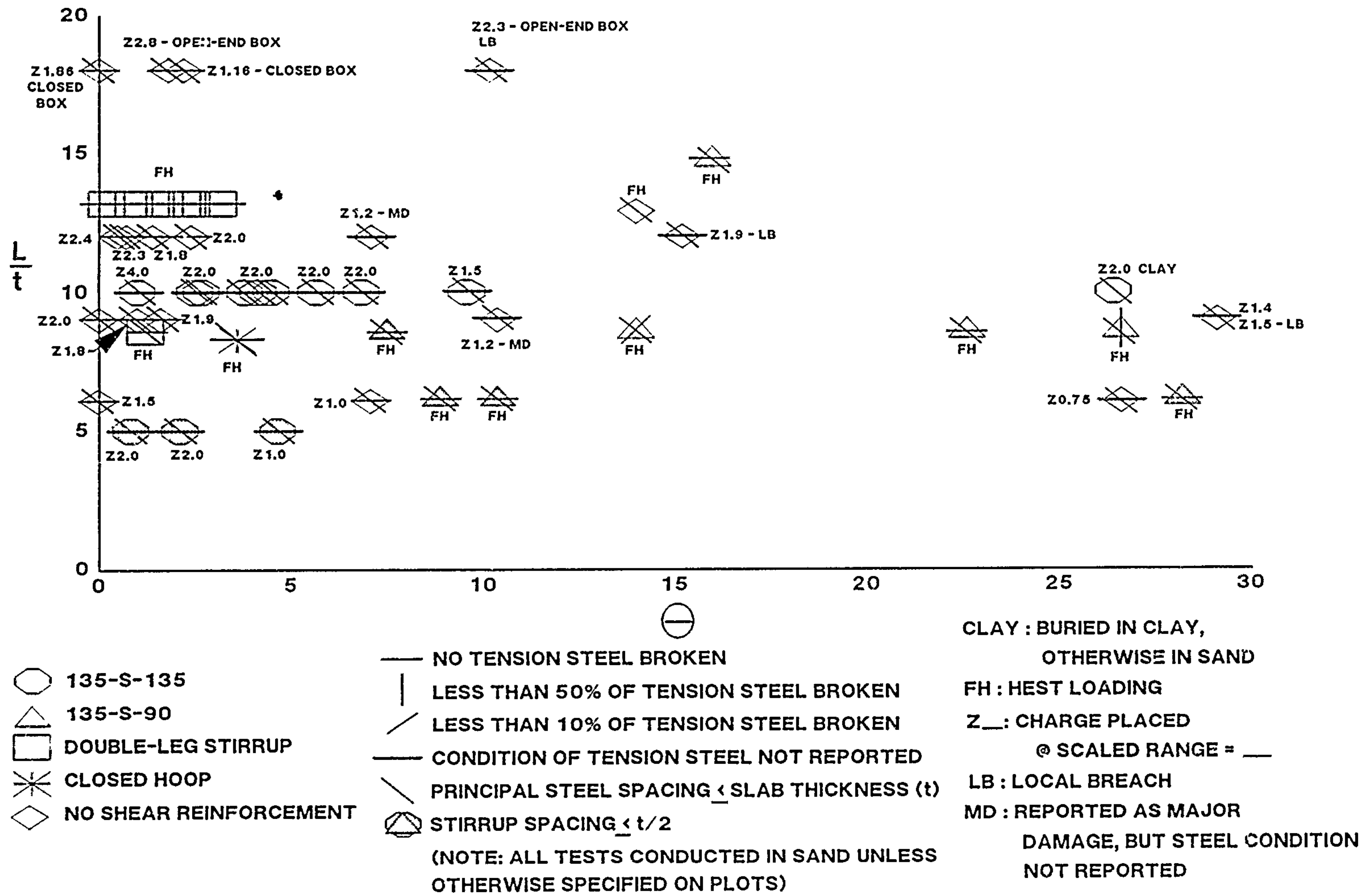


Figure 14. Dynamically-Tested Buried Boxes

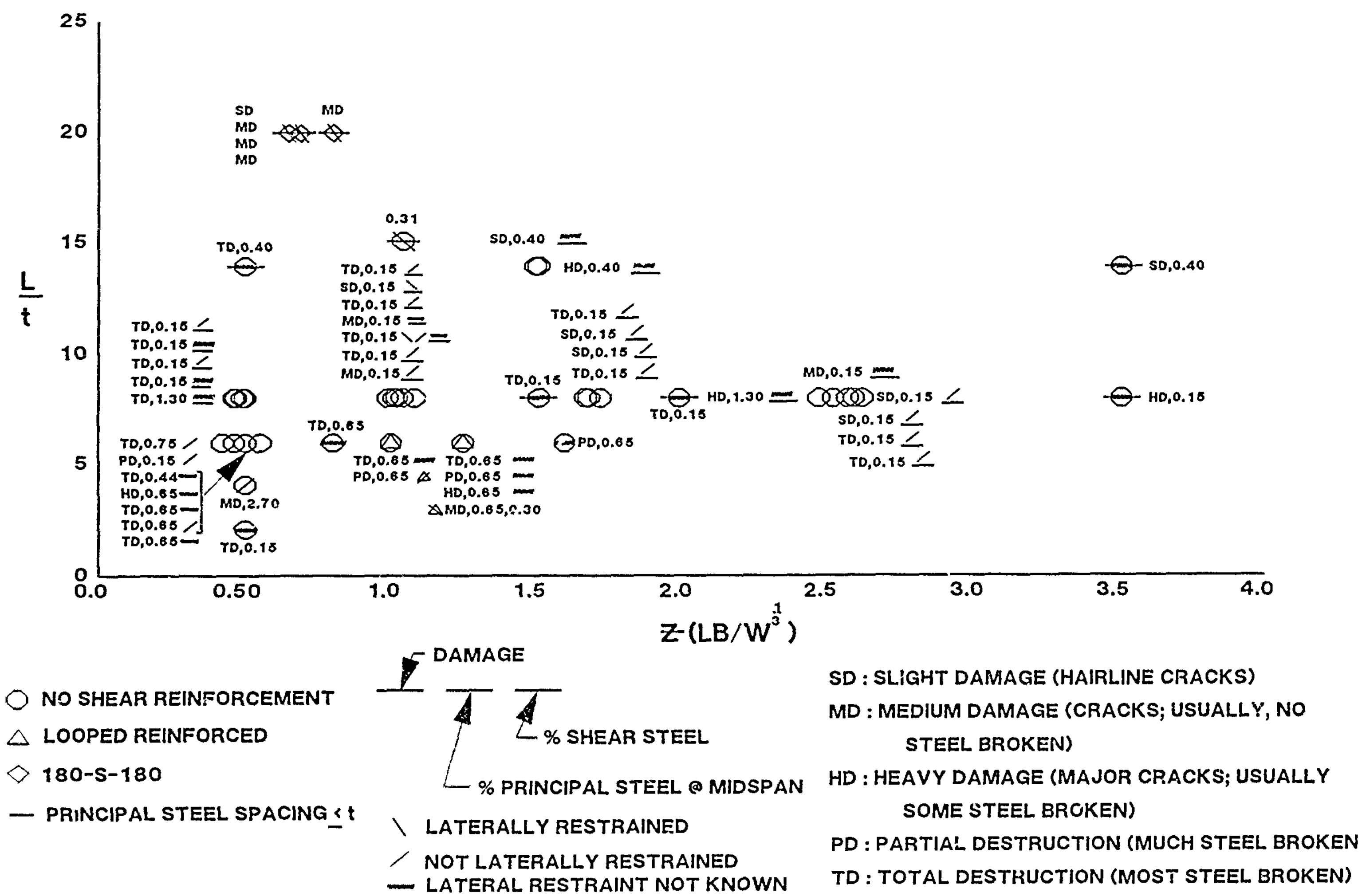


Figure 15. Dynamicall,-Tested Non-Laced Slabs

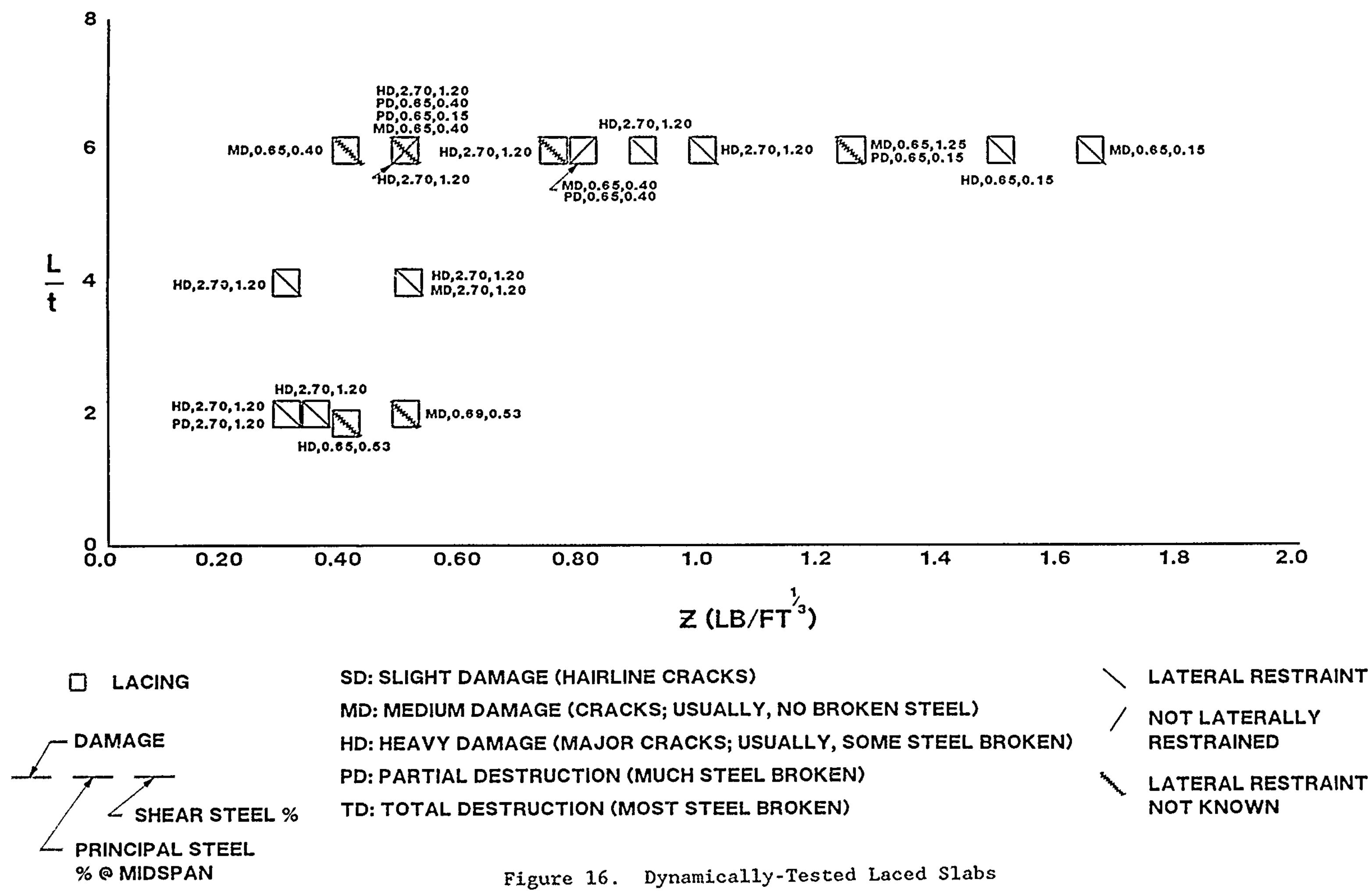


Figure 16. Dynamically-Tested Laced Slabs